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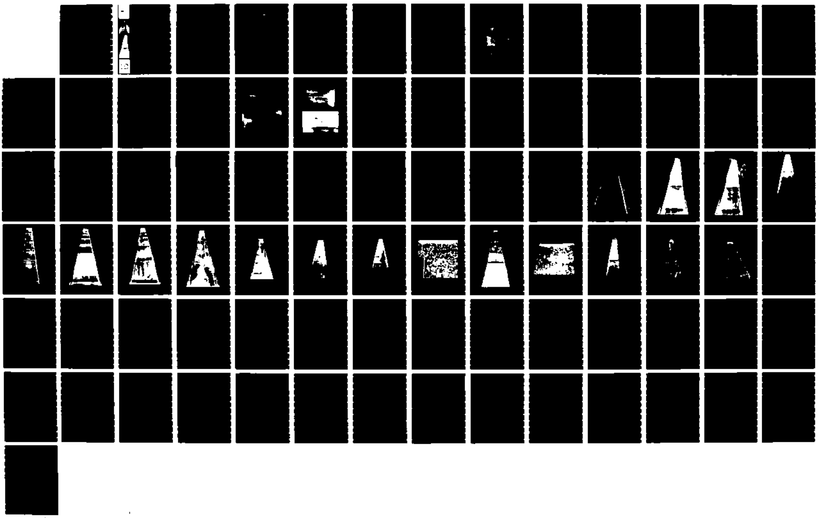

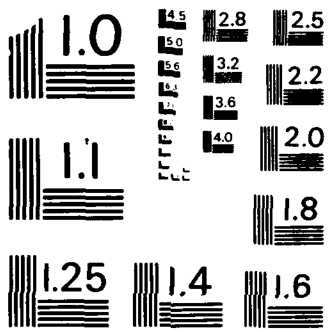

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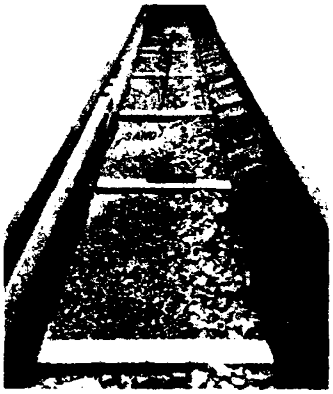

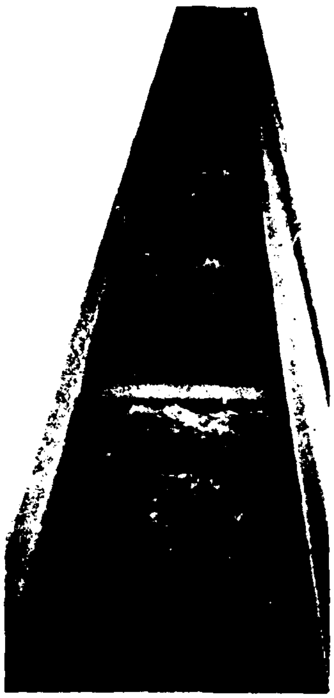

HYDRAULICS LABORATORY

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TECHNICAL REPORT HL-86-2

MILL CREEK CHANNEL, WALLA WALLA, WASHINGTON

Hydraulic Model Investigation

by

Deborah W. Robinson, Ronald R. Copeland

Hydraulics Laboratory

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
PO Box 631, Vicksburg, Mississippi 39180-0631

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US ARMY ENGINEER WATERWAYS EXPERIMENT STATION

May 1986
Final Report

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A 1:10-scale hydraulic model simulated a 40-ft-wide section of the channel with five stabilizers in a sand-gravel bed stream. Scour depths downstream from the stabilizers were observed and riprap protection was recommended. Riprap protection along the side slopes was also recommended.

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PREFACE

The model investigation of the Mill Creek channel was authorized by the US Army Engineer District, Walla Walla (NPW), on 19 July 1982.

This investigation was conducted during the period July 1982 to June 1983 in the Hydraulics Laboratory at the US Army Engineer Waterways Experiment Station (WES), under the direction of Messrs. H. B. Simmons and F. A. Herrmann, Jr., former and present Chiefs of the Hydraulics Laboratory, and Mr. J. L. Grace, Jr., Chief of the Hydraulics Structures Division, and under the general supervision of Mr. N. R. Oswalt, Chief of the Spillways and Channels Branch. Project Engineers for the model study were Ms. D. W. Robinson and Mr. R. R. Copeland, who also prepared this report. Mr. E. L. Jefferson, Technician, assisted in the study. This report was edited by Mrs. Beth F. Vavra, Publications and Graphic Arts Division.

During the course of the study, Mr. Bill Branch of NPW, Mr. John Oliver of the North Pacific Division, and Mr. Tom Munsey of the Office, Chief of Engineers, visited WES to discuss the program of model tests, observe the model in operation, and correlate test results with concurrent design work.

Director of WES was COL Allen F. Grum, USA. Technical Director was Dr. Robert W. Whalin.

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CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
cubic feet per second	0.02831685	cubic metres per second
feet	0.3048	metres
inches	2.54	centimetres
miles (US statute)	1.609344	kilometres
pounds (mass)	0.4535924	kilograms

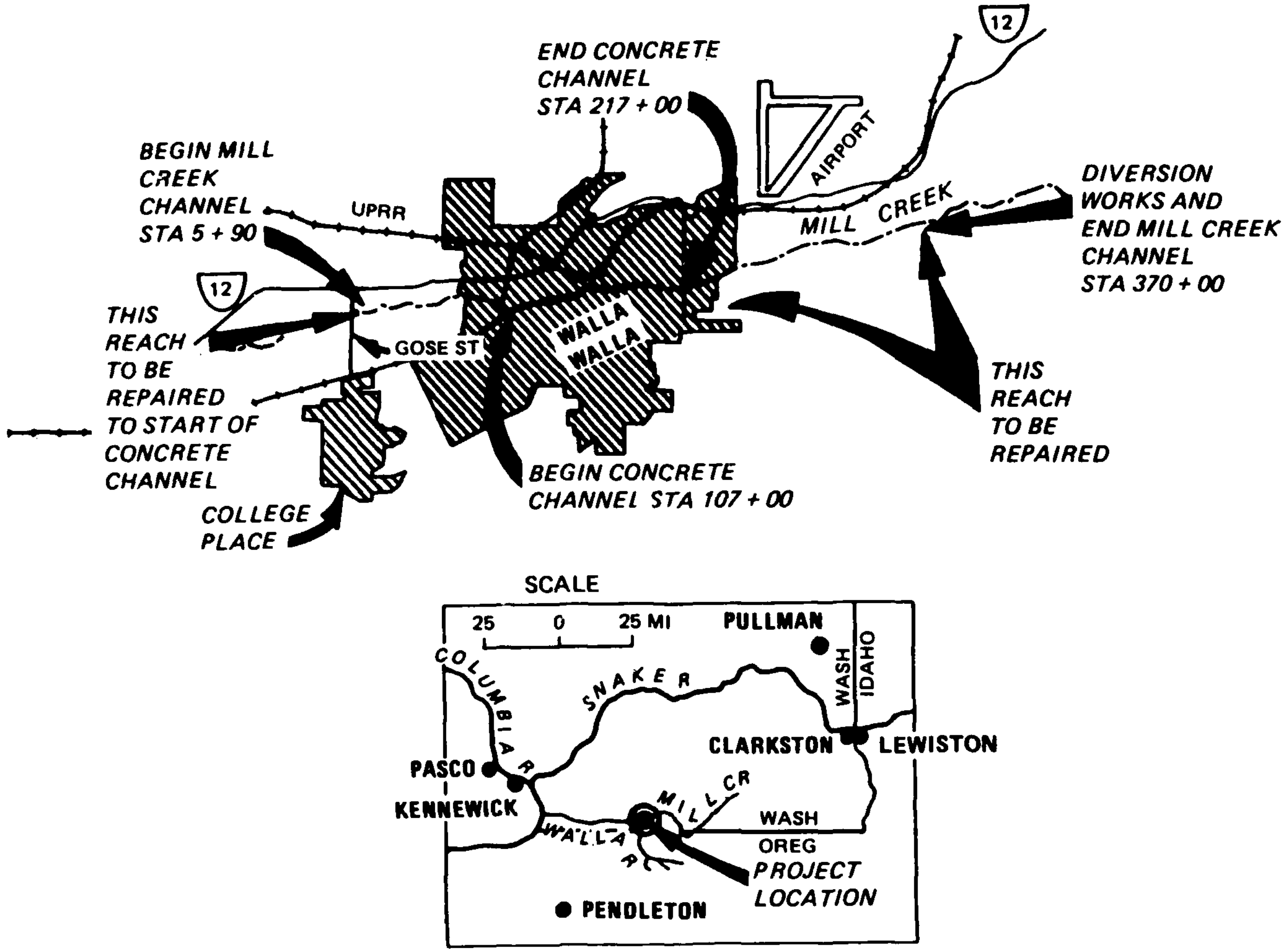

Figure 1. Location map

MILL CREEK CHANNEL, WALLA WALLA, WASHINGTON

Hydraulic Model Investigation

PART I: INTRODUCTION

The Prototype

1. Mill Creek is located in southwest Washington and runs through the city of Walla Walla (Figure 1). The Mill Creek Flood Control Project was designed to provide flood protection to the city, irrigation to adjacent farmland, and passage for migrating fish. A diversion dam is located at the upstream end of the project that can be used to divert portions of peak flood flows to an offstream storage reservoir. Mill Creek itself is improved for about 6.9 miles* downstream from the dam. The stream slope varies from 0.016 at the diversion dam to 0.010 at Gose Street. The bed material in the creek is sand and gravel. Between the diversion dam and Roosevelt Street, about 2.9 miles, the channel has wirebound gravel mattresses on the banks and concrete-capped wirebound gravel stabilizers on the bottom. Stabilizers have cutoff walls with base elevations about 5.5 ft below the crest. Stabilizer spacing is generally between 60 and 70 ft. Downstream from Roosevelt Street, for about 2.1 miles through the city of Walla Walla, Mill Creek is concrete-lined with vertical sidewalls, a sloping floor, and a pilot channel with baffles in the center of the floor for upstream migration of fish. The remaining 1.9 miles consists primarily of uncapped wirebound gravel stabilizers with wirebound gravel mattresses on the bank.

2. From the time of construction of the improved Mill Creek channel in the late 1930's, the stabilizers and the mattresses have experienced various structural difficulties. The wire on the uncapped stabilizers has deteriorated to a point that some stabilizers have washed away. Many of the remaining stabilizers are in need of repair. Problems with the mattress bank protection stabilizers are generally related to broken wire and undercutting in plunge pools downstream from stabilizers. Plunge pools that form during floods have sometimes been filled by bulldozing material from downstream.

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* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page [illegible].

Subsequent floods tend to reestablish the plunge pool.

Purpose of the Model Study

3. The ability of Mill Creek channel to contain the Standard Project Flood of 5,400 cfs was questioned by engineers from the Walla Walla District. The maximum discharge that the stabilizers could pass without failure was unknown. Design guidance for protecting existing stabilizers and repairing damaged ones was also needed. The US Army Engineer Waterways Experiment Station (WES) was asked to provide engineering assistance.

4. A limited numerical model study was conducted at WES to evaluate the bed's stability at various discharges. The HEC-6 computer program "Scour and Deposition in Rivers and Reservoirs" was used for this analysis. This program is not designed to consider local scour problems, e.g., downstream from stabilizers, but can be used to evaluate general scour and bed stability. Bed profiles computed with the numerical model are compared with those determined with the physical model for the 1945 flood hydrograph in Figure 2. Although

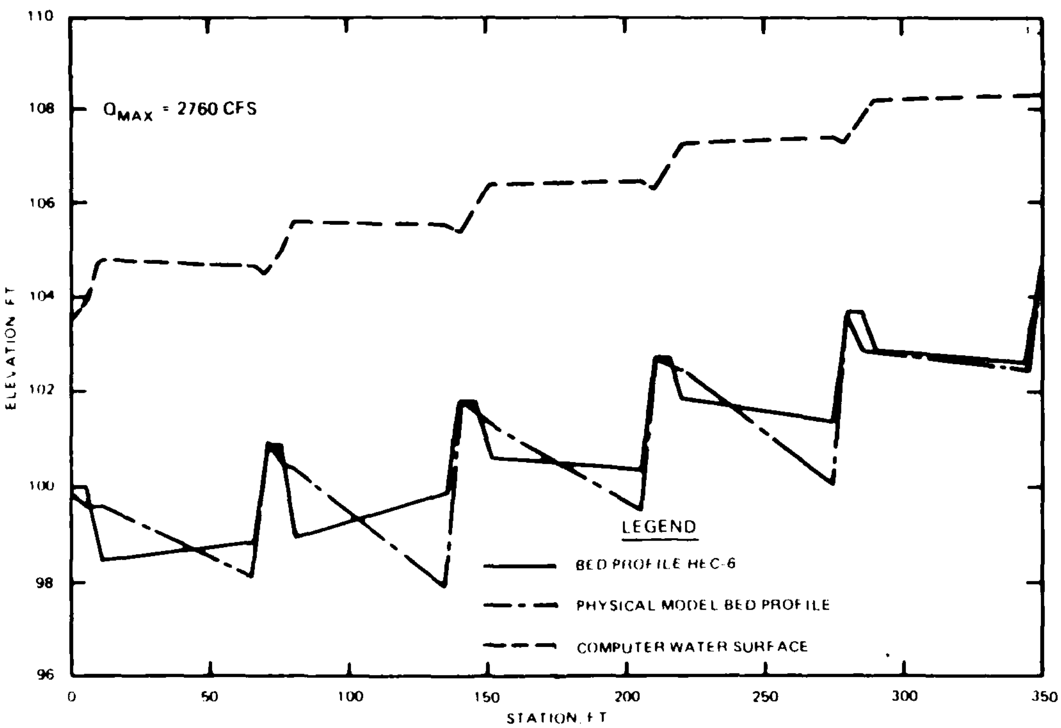

Figure 2. Comparison of HEC-6 and physical model bed profiles, 1945 hydrograph

the numerical model was able to adequately predict scour volumes, it could not pick up the variation in bed profile caused by local scour. The conclusion of the numerical model study was that the stability of channel stabilizers above 3,000 cfs was questionable. A physical model study was recommended.

5. The physical model study was conducted to provide design guidance for the stabilizers and banks of Mill Creek channel. Scour potential downstream from existing stabilizers and at the channel bank was determined. The model was used to determine adequate protection for the existing concrete-capped stabilizers and for proposed sheet-pile stabilizers. Adequate riprap sizing and toe depths were determined for bank protection. The model was used to help determine the design discharge for Mill Creek.

PART II: THE MODEL

Description

6. A 1:10-scale section model was used to simulate a 40-ft-wide section of Mill Creek channel (Figure 3). Two reaches of the channel were

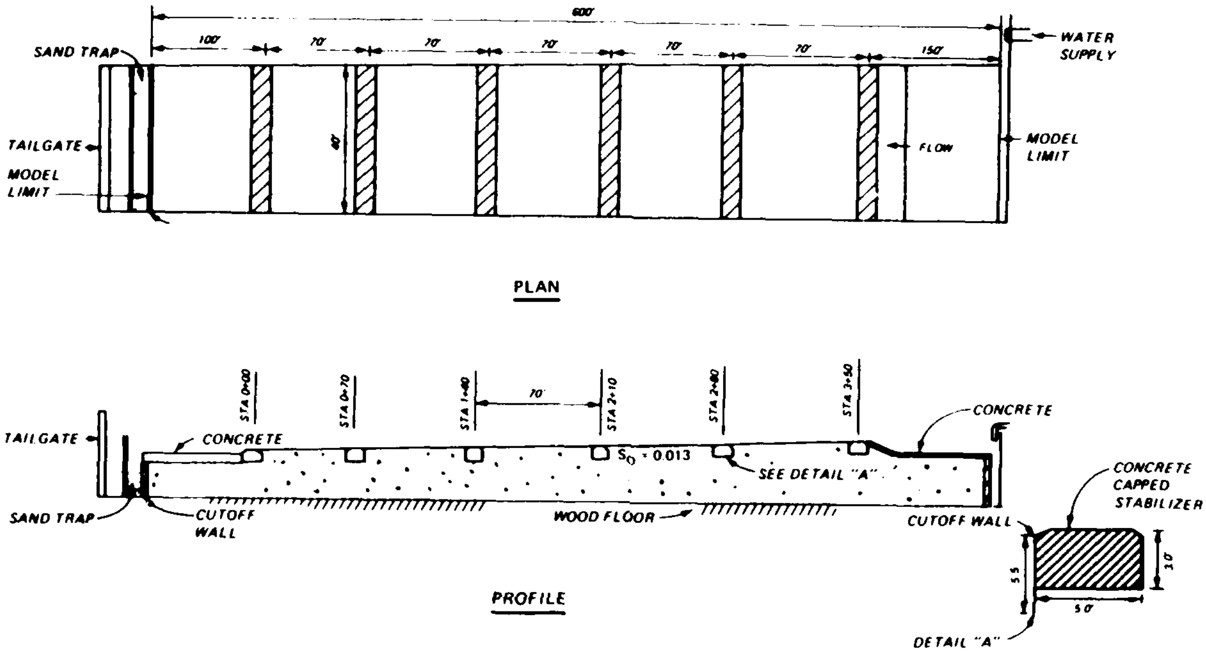

Figure 3. 1:10-scale model, 70-ft simulated width

investigated. Where the prototype channel is 70 ft wide and stabilizers are 70 ft apart, 5 stabilizers were simulated; where the prototype channel is 120 ft wide and stabilizers are 60 ft apart, 6 stabilizers were simulated. Concrete stabilizers were simulated using wood that was painted with enamel to obtain a smooth finish. The 5.5-ft cutoff walls were simulated with sheet-metal strips attached to the front of the stabilizers. Sheet-metal stabilizers were simulated using noncorrugated sheet metal. Rock and wire mattresses and fascines used on side slopes were simulated using standard aluminum screen filled with pea gravel. The bed material was obtained by mixing rounded gravels and sand to obtain a gradation that simulated the coarser 70 percent of the prototype gradation. Exact reproduction of the finer portion was not attempted in this case to avoid problems relating to cohesive forces that would have occurred if the fine sand sizes in the prototype had been scaled down in the model. However, this finer 30 percent of the

material should react similarly in the model and prototype as it will be transported through both channels unless covered by an armor layer. Comparison of model and prototype bed gradations is shown in Plate 1.

7. The 4-ft-wide flume used in the study was 60 ft long of which 15 ft was used as approach channel and 10 ft was exit channel. The approach and exit channels were paved with rough concrete approximating prototype bed profiles. In tests with side slopes, the correct cross-sectional areas were maintained in the approach and exit channels using grouted-rock side slopes.

8. Water was supplied to the flume from a constant-head tank and was regulated using venturi meters. Water entered the flume from a headway where a constant head was maintained, ensuring a uniform flow distribution in the flume. Tailwater elevation was regulated by a tailgate located at the downstream end of the flume. Sediment was not recirculated in the flume.

9. Water-surface elevations were measured by means of a point gage that slid along a level horizontal bar. Bed profiles were measured with a sounding rod having a 1-in. swivel disk on the bottom that smoothed out local irregularities in bed elevations.

Interpretation of Model Results

10. The flow patterns in Mill Creek channel are affected by two dominant forces--gravity and inertia. Dynamic similarity is achieved in the model by applying the accepted equations of hydraulic similitude, based on the Froudian criteria. The following relations are valid for transferring model results to prototype equivalents:

Characteristic	Ratio	Scale Relation
Length	L_r	1:10
Area	$A_r = L_r^2$	1:100
Velocity	$V_r = L_r^{1/2}$	1:3.162
Discharge	$Q_r = L_r^{5/2}$	1:316.2
Time	$T_r = L_r^{1/2}$	1:3.162
Weight	$W_r = L_r^3$	1:1,000

Forces due to viscosity, surface tension, and elasticity can also influence

the hydrau'ic characteristics of the prototype and model. In this model study, Reynolds numbers were sufficiently large (1×10^5) to render the effects of these forces negligible with respect to flow of water.

11. Applying Froudian criteria to the entrainment and transport of sediment can introduce scale problems. Sediment transport is influenced somewhat by viscous forces even at high Reynolds numbers. Size and weight simulation of fine sand in the prototype cannot be achieved in the model without introducing error due to the cohesiveness of silts and clays. Thirty percent of the bed material in Mill Creek fell into this category. However, this was not considered critical because this class of material should behave as wash load in both the model and prototype. That is, due to the small size of the material, once the armor layer above it is removed these particles should be entrained and transported out of the model in suspension. Nevertheless, it is essential to verify the physical model's ability to predict scour depths. Verification was accomplished for the Mill Creek channel using available prototype data.

PART III: TEST RESULTS

Method of Operation

12. The following procedure was used for each model test. The model was flooded from the downstream end with the tailgate raised. Once a pool was established and the air that was trapped in the model bed was released, the desired discharge was set and the tailgate gradually lowered. Timing of the discharge duration started when the tailgate was completely lowered. The discharges and duration of given hydrographs were simulated. The duration of each discharge was shortened if there was no change in the bed profile during a given time interval. Observations of the model bed were made through a glass side of the flume during each test. Bed and water-surface profiles were measured at the end of each flow duration. The bed was remolded after the completion of each hydrograph.

Model Verification

13. The section of Mill Creek with concrete-capped stabilizers spaced 70 ft apart was chosen for the verification tests. The model was constructed to simulate prototype channel conditions immediately after construction (Photo 1). The channel had a constant slope of 0.013, with the tops of the stabilizers at grade. The streambed was not armored. The model was verified by running three historical flood hydrographs and comparing model and prototype scour depths.

14. Several small flows occurred in the channel between the completion of construction and the 1945 flood event. Flows of 400, 900, and 700 cfs (representing actual high mean daily flows) were run through the model preceding the 1945 flood hydrograph. At 400 cfs, considerable transport of fines occurred and an armor layer was established.* A local scour hole developed downstream of each stabilizer. Flow became more turbulent when the discharge was increased to 900 cfs, the initial armor layer was destroyed, deeper local scour holes developed, and a coarser armor layer was established. When

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* The armor layer developed as fine particles were washed away leaving behind a thin layer of immobile coarser material.

700 cfs was run through the model the bed profile remained essentially unchanged. Bed profiles recorded during these discharges are shown in Plate 2.

15. The 1945 flood hydrograph was then run through the model (Plate 3). When the discharge was increased to 1,600 cfs the armor layer established at 900 cfs was destroyed, and sediment transport and local scour downstream from the stabilizers increased. Flow was increased to the peak of 2,760 cfs and held constant for 2 hr (prototype time). Bed material was still being transferred when the recession of flow began. During falling stages the armor layer, which had developed at the peak flow, remained stable. The finer fractions of the bed material that had accumulated immediately downstream from the stabilizers at high flows were scoured away from the structures at lower flows. This occurred because at lower discharges the flow trajectory over the stabilizers decreased and at very low discharges fell vertically down the face of the structure. The result was a deep scour hole adjacent to the structure. Typical bed profiles observed with the 1945 hydrograph are shown in Plate 4.

16. The 1946 flood hydrograph with only a peak flow of 1,600 cfs was used to model-test the armor layer established during the 1945 flood. No movement in the armor layer was observed as was expected with the lesser rate of flow. Fines in the local scour hole, overlying the armor layer, were recirculated back against the stabilizers and rearranged to fill in immediately downstream of the stabilizer with increased discharges. Fines were moved away from the stabilizer and deposited downstream as the flow receded. Typical bed profiles observed are provided in Plate 5.

17. Prototype maintenance work was simulated in the model followed by the 1982 flood hydrograph with a peak discharge of 1,730 cfs. The bed of the model was raked to simulate the scarification and regrading of existing bed material accomplished by means of a bulldozer in the prototype after the prototype has experienced scour sufficient to justify maintenance. One 70-ft-long reach was completely raked and the other four sections were raked within a 35-ft-long reach immediately downstream of the stabilizers. A distinct line of separation was observed through the glass-sided flume between the coarser armored layer of the previously formed local scour hole and the finer bed material raked into the scour hole. The raking weakened the armor and more transport and scour were observed in subsequent tests using the 1982 hydrograph.

Thus it is considered that placement of additional riprap or armor material downstream of each stabilizer would be more appropriate than scarification and regrading of existing bed material in Mill Creek.

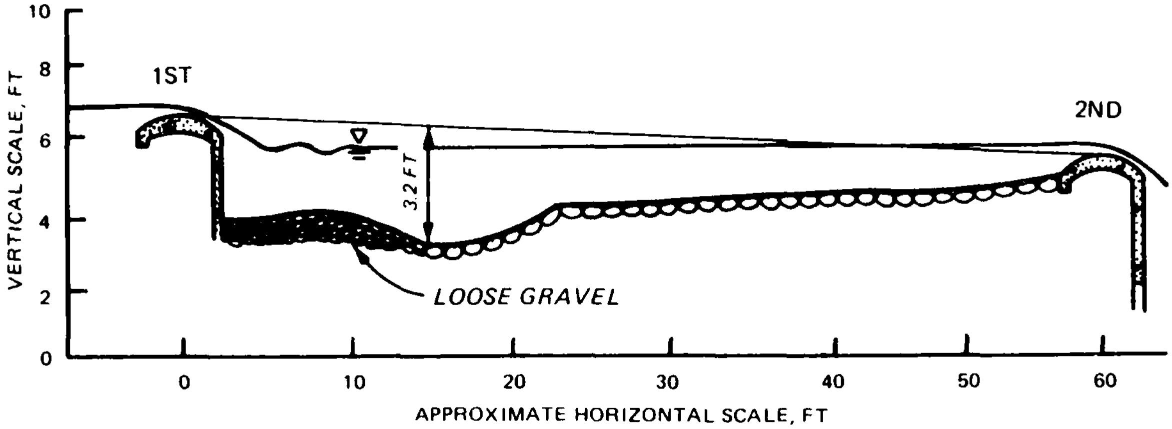
18. Comparisons between measured and simulated scour depths is difficult due to the general lack of historical data. A prototype survey of a reach of the 70-ft-wide channel with concrete-capped stabilizers was conducted in 1980. This survey revealed that plunge pools ranged in depth from 0 to 5 ft, with over 60 percent about 3 ft deep. Scour depths ranged between 3.2 and 4.0 ft in the physical model after the 1945 hydrograph. These results are not directly comparable because significant channel rehabilitation work occurred in the channel between 1945 and 1980. This work included filling of plunge pools with material from downstream, subsequently destroying the established armor layer. After the 1982 flood (Q = 1,730 cfs), prototype scour depths of 4.8, 3.5, and 3.1 ft were measured downstream of three stabilizers in the 120-ft-wide reach and a prototype scour depth of 3.2 ft was measured in a 75-ft-wide section. For comparison, scour depths ranging between 2.3 and 3.2 ft were observed in the physical model after the 1982 hydrograph. This flood was simulated over a rehabilitated streambed (raked in the model, bulldozed in the prototype) in a 70-ft-wide reach. Loose gravel piles generally located between the stabilizer and the deepest point of the scour hole were observed in both the model and the prototype (Figure 4). End views of the prototype and model channel beds, looking upstream, are shown in Figure 5.

19. Samples of the prototype armor layer were compared with the armor layer that developed in the physical model and are shown in Figure 6.

20. A meeting was held in the Walla Walla District attended by engineers from WES and the District to evaluate model performance. After presenting the results of the verification tests, the District was satisfied that the model was adequately simulating the prototype.

Existing Channel Designs

21. Tests were conducted to observe general flow conditions and determine the adequacy of the existing channel. Initial tests were conducted to determine a design discharge for which the channel could be adequately protected. For these tests, a 350-ft-long section of the 70-ft-wide channel of Mill Creek was simulated (40 ft of the width was simulated in the section



a. Prototype stabilizers at sta 317+70 and 318+30, 17 Jun 1982, channel width 75 ft

b. Model scour hole downstream from stabilizer after Feb 1982 flood hydrograph, 70-ft simulated channel width

Figure 4. Scour downstream from concrete-capped stabilizer after 1982 flood hydrograph

a. Prototype: sta 235+85, Sep 1980

b. Model: after verification tests

Figure 5. End view of channel bed, looking upstream

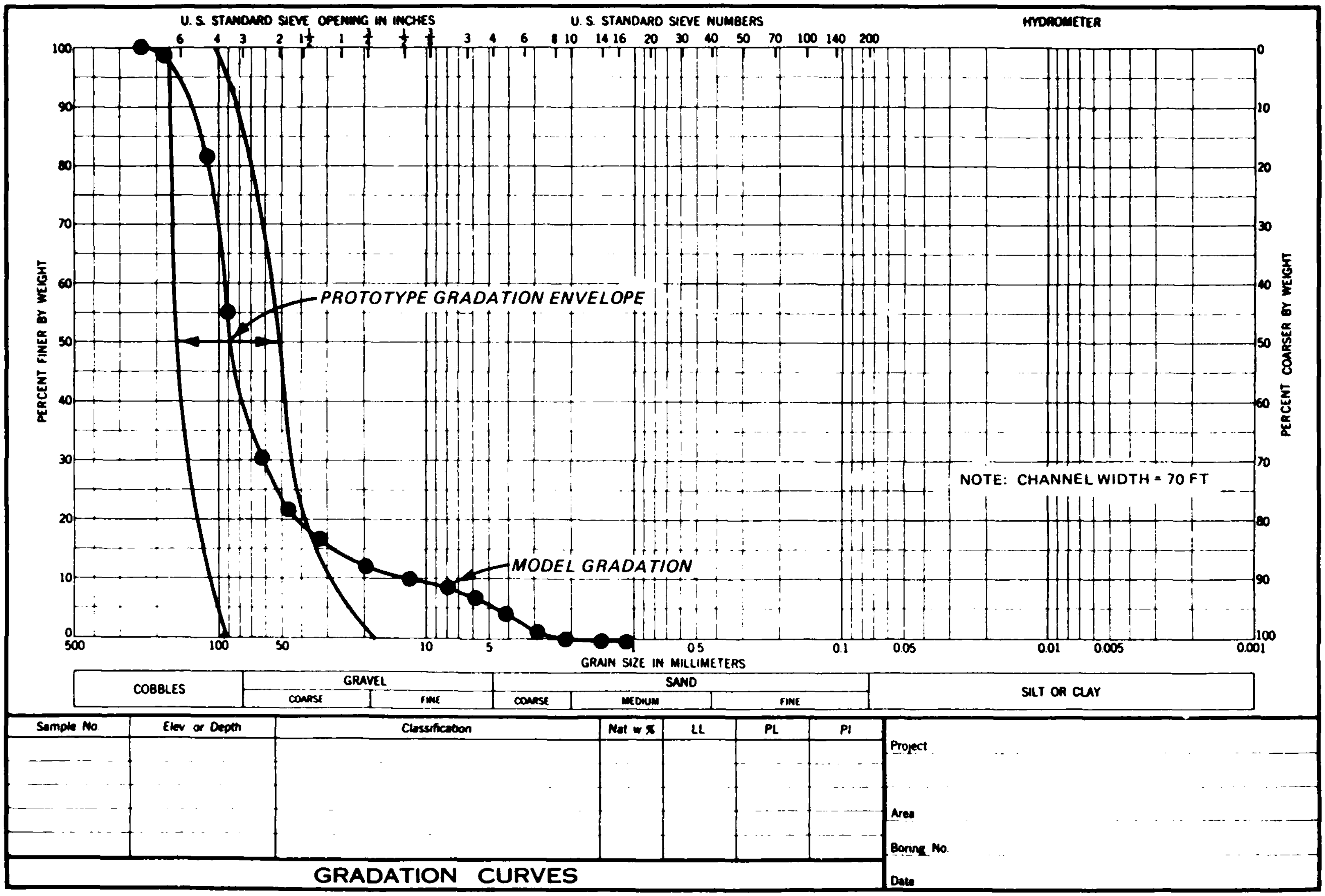


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Figure 6. Armor layer gradation curves

model). The bed was molded to a 0.013 ft/ft slope between six concrete-capped stabilizers spaced 70 ft apart (Photo 1).

22. Flow conditions and scour development with peak discharges ranging from 3,000 (Photo 2) to 5,000 cfs* (Photo 3) were observed in the channel without protection downstream of the stabilizers. The duration for the peak discharge was originally 24 hr but was shortened to 19 hr once it was observed that no discernible change occurred in the scour pattern. Plates 6-10 are bed and water-surface profiles for discharges of 3,000, 3,500, 4,000, 4,500, and 5,000 cfs, respectively. A design discharge of 3,500 cfs was chosen because this discharge produced scour depths that fell into the range of allowable scour depths of 4 to 5.5 ft provided by the Walla Walla District.

23. The maximum scour depth produced by the design discharge of 3,500 cfs was 5.6 ft; the average scour depth was 5.1 ft (Plate 7). Initially, transport of material and scour development were rapid with most of the scour occurring during the first 3 hr and increasing 0.3 to 0.4 ft in the average scour depth in the next 16 hr. The armor layer was established during the first 3 hr. Most of the movement after the establishment of the armor layer was in the area of the scour hole. The upstream and downstream sections in this and subsequent tests were not used in computing average scour depths, because they were not considered representative due to model boundary effects. The armor layer established at the peak discharge of 3,500 cfs was not affected by the lower flows on the recession limb of the hydrograph, nor did the flow going over and down the stabilizer face undermine the structure or deepen the scour hole. It did, however, move fine material that accumulated at the face of the stabilizer downstream.

Concrete-Capped Stabilizer Design, 70-ft-Wide Channel

24. Tests were conducted to develop an adequate apron design downstream from the concrete-capped stabilizers in the 70-ft-wide channel. The apron was deemed necessary because the scour hole could eventually move upstream and undermine the stabilizer during low-flow events. Locally available riprap is classified Class A (200 lb) and Class B (60 lb); gradations are provided in

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* A table of unit discharges for the given discharges is presented in Table 1.

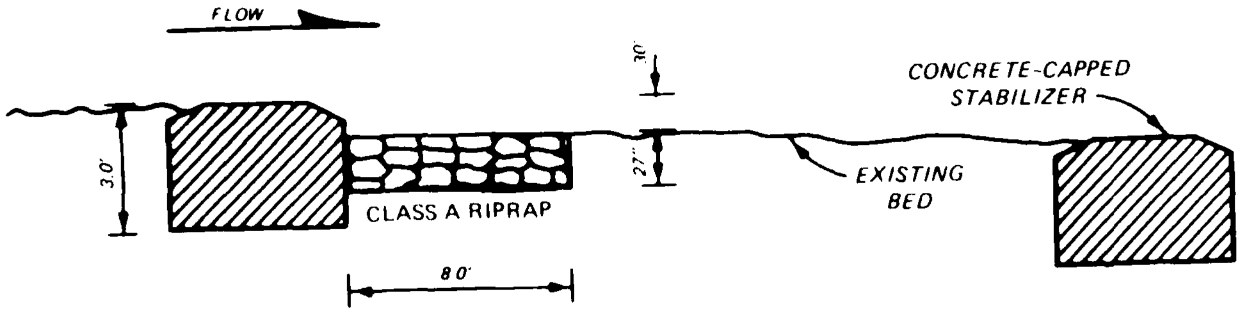

Figure 7. Type 1 (original) design stabilizer protection

Table 2 and Plate 11. The original Type 1 design stabilizer protection (Figure 7) was an 8-ft-long, 27-in.-thick horizontal apron composed of Class A riprap, the top of which was located 30 in. below the crest of the stabilizer. This design was tested using 3,000- and 4,000-cfs hydrographs (the design discharge of 3,500 cfs had not been chosen at this stage of testing). These hydrographs produced average scour depths of 4.7 and 5.6 ft, respectively (Plates 12 and 13). The riprap remained relatively intact during the 3,000-cfs hydrograph, but there was considerable movement and separation of the protection during the 4,000-cfs hydrograph. Before-and-after shots of the Type 1 design stability protection with the 4,000-cfs hydrograph are shown in Photos 4 and 5.

25. Stability was improved with the Type 2 design stabilizer protection (Figure 8) by sloping the Class A riprap down on a 1V-on-3.1H slope. For the Type 2 design stabilizer two different initial bed profiles were tested. The bed initially was molded to the top of the riprap toe as shown by (A) in Figure 8 and for subsequent tests the initial bed was molded to cover the riprap completely downstream of the stabilizer as shown by (B) in Figure 8. The 3,500-cfs design hydrograph was used to test this design. Maximum scour depth was 5.6 ft; average scour depth was 5.3 ft (Plate 14). The type (A) initial bed profile yielded scour depths greater than (B). The design proved to be more stable because it conformed with the slope of the scour hole with both initial bed profiles. As material was transported over the stabilizers, the

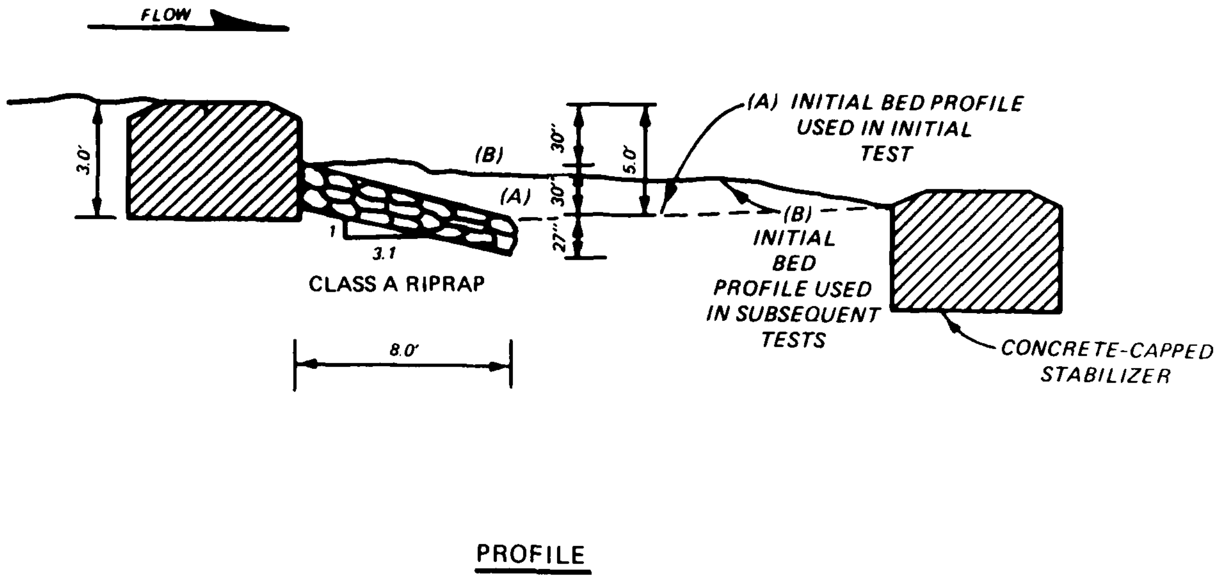

Figure 8. Type 2 design stabilizer protection

fines were deposited on top of the riprap protection, filling in the voids and providing added strength to the riprap protection. The bed downstream of the scour hole remained stable after 3 hr with all the bed movement occurring in and around the scour hole for the duration of this hydrograph. The Type 2 design stabilizer protection was recommended for the concrete-capped stabilizers in the 70-ft-wide channel.

Sheet-Pile Stabilizers Design

26. The six concrete-capped stabilizers were replaced with six sheet pile stabilizers 6 ft deep and 70 ft apart. The bed slope remained 0.013 ft/ft. This Type 3 design stabilizer is shown in Figure 9 and Photo 6. This design consisted of the same downstream apron used in the Type 2 design and a 5-ft-long, 18-in.-thick upstream apron consisting of Class B riprap. Two initial bed profiles were tested. The bed was molded to the top of the riprap as shown by (A) in Figure 9 and for subsequent tests the initial bed was molded to cover the riprap completely as shown by (B) in Figure 9. The initial bed profile (A) produced maximum scour depths. The top of the upstream apron was located at the stabilizer crest in all tests using the Type 3 design. The 3,500-cfs design hydrograph with the Type 3 design stabilizer produced a maximum scour depth of 4.5 ft and an average depth of 4.3 ft (Plate 15).

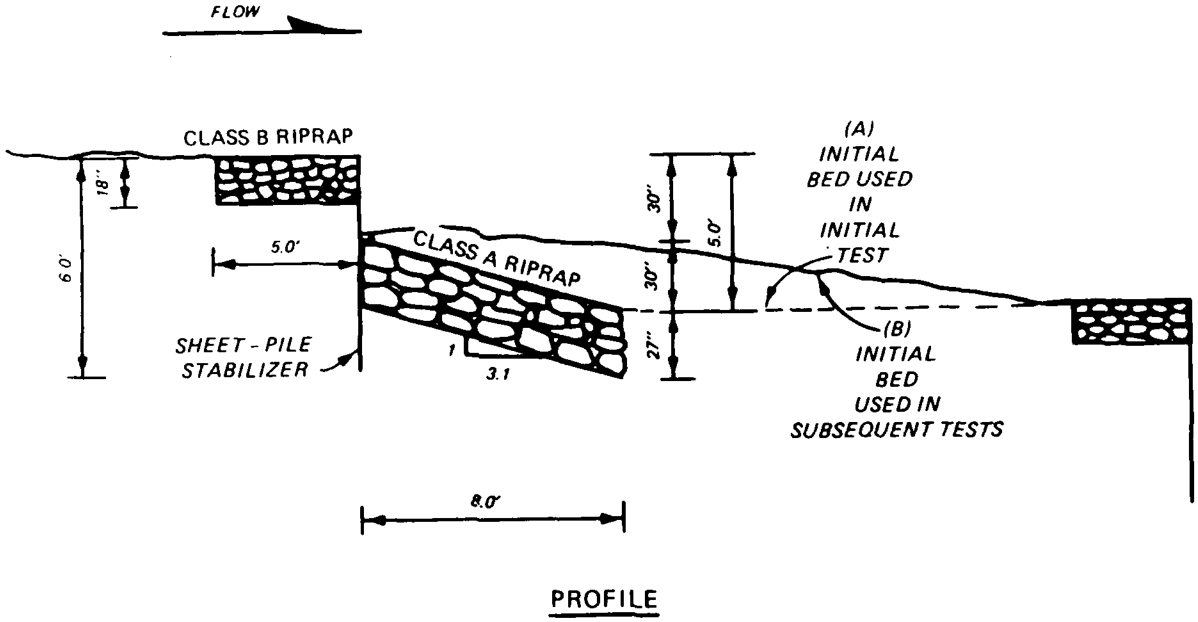

Figure 9. Type 3 design stabilizer protection

Fines covered the riprap downstream of the stabilizer (Photo 7), and there was fluttering of the upstream riprap. Because of this fluttering, the design was considered to be unstable.

27. To remedy the riprap fluttering on the crest, the upstream riprap was recessed 0.5 ft from the top of the sheet piling, and placed on a 1V-on-6H slope in the Type 4 design stabilizer (Figure 10). The downstream riprap remained the same. The Type 5 design stabilizer (Figure 11), in which the

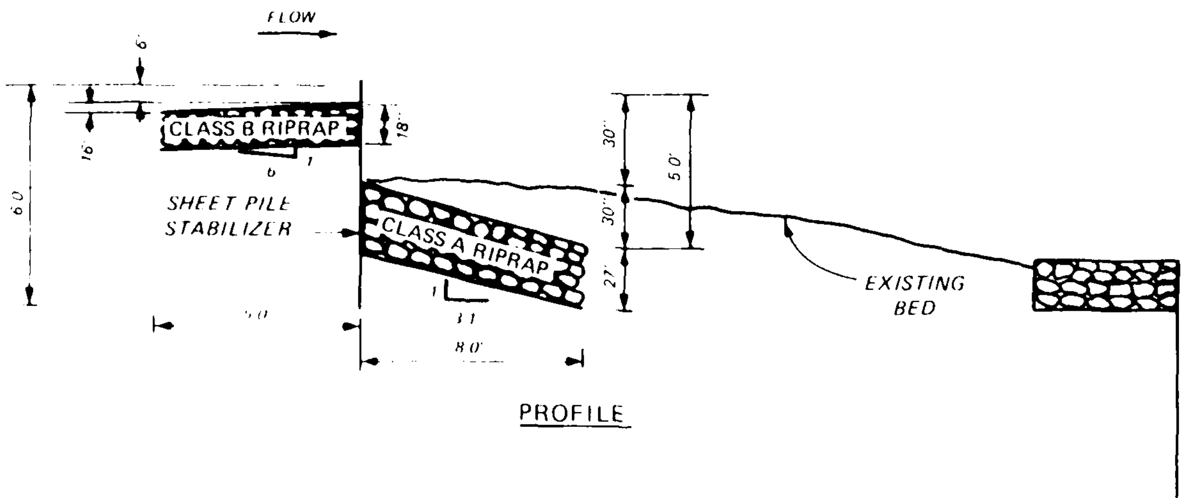

Figure 10. Type 4 design stabilizer protection

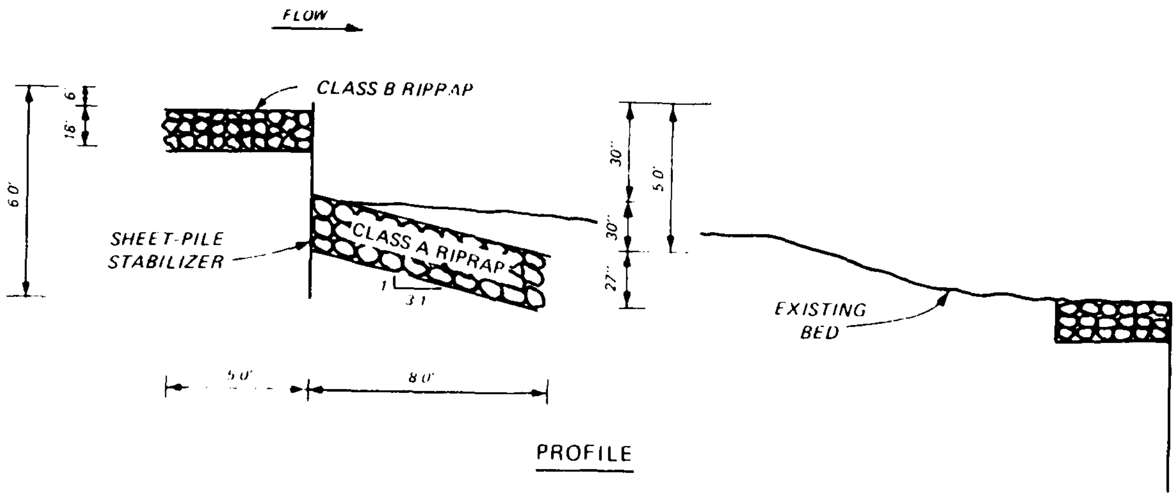

Figure 11. Type 5 design stabilizer protection

upstream riprap was recessed 0.5 ft below the top of the sheet pile but remained horizontal, was alternated with the Type 4 design stabilizer in the model as indicated in Plates 16 and 17. A peak discharge of 3,000 cfs (Plate 16) produced a maximum scour depth of 4.1 ft and an average of 3.6 ft. The design discharge of 3,500 cfs (Plate 17) produced a maximum scour depth of 4.6 ft and an average of 3.5 ft. There was no discernible difference in scour with the Type 4 and 5 design stabilizers. Types 4 and 5 remained intact in both tests; but Type 4 proved more stable, because of the manner in which the bed material filled in the voids of the upstream riprap protection. The Type 4 design was recommended for sheet-pile stabilizers.

28. The effect of staggering the sheet-pile heights was tested using the Type 6 design stabilizer (Figure 12). Heights on alternate sheet piles were increased by 0.5 ft creating notches 1.17 ft wide. The riprap used in the Type 4 design stabilizer was retained. Maximum scour depth decreased to 4.4 ft and the average scour depth increased to 3.9 ft, while causing no significant change in the

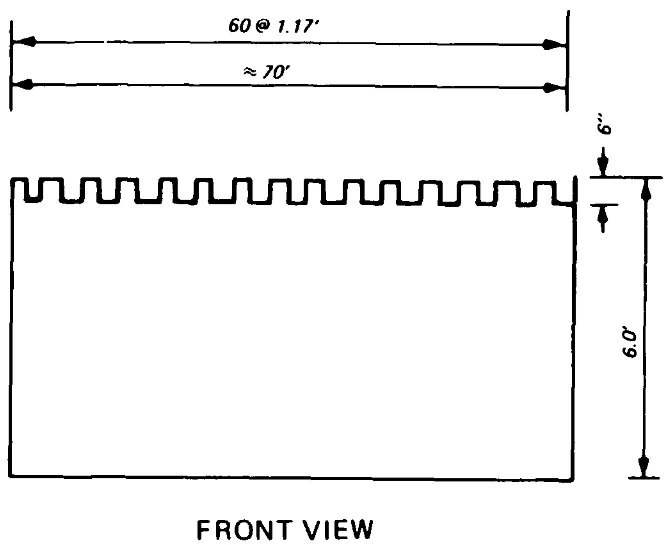

Figure 12. Type 6 design stabilizer

tailwater depth (Plate 18). No apparent benefit was observed with the Type 6 design stabilizer.

Bank Toe Protection Design (Type 1)

29. The original Type 1 design bank protection (Figure 13) was tested

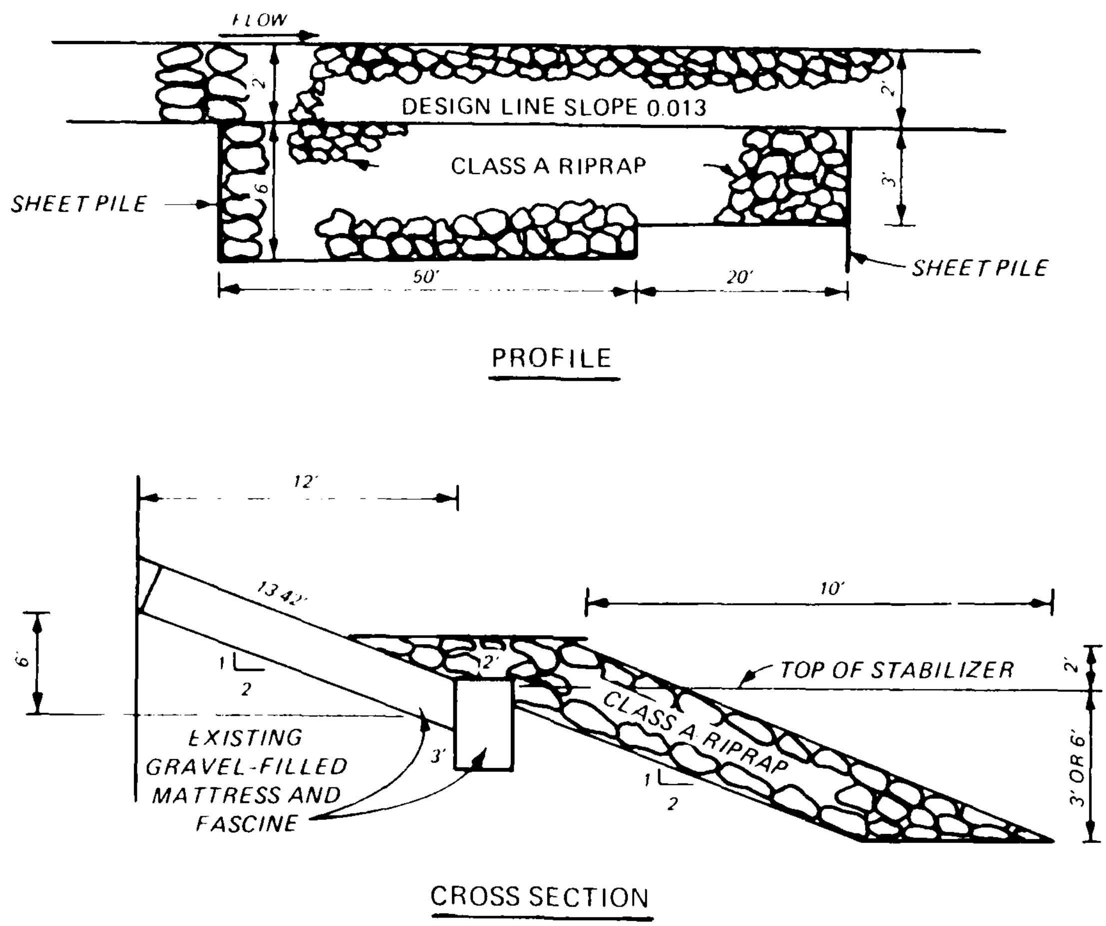

Figure 13. Type 1 (original) bank toe protection

for the sheet-pile and concrete-capped stabilizers (Photo 8). Bank protection consisted of Class A riprap placed on a 1V-on-2H slope. Toe depths were 6 ft for 50 ft downstream from the stabilizer and 3 ft for the remaining 20 ft. The riprap was placed over the existing wirebound gravel mattresses and fascines up to 2 ft above the design grade line. Peak discharges of 3,000 and 3,500 cfs were run through the model. Maximum scour depth occurred along the toe of the slope where the channel bed and bank protection intersected,

because the hydraulic jump was concentrated at the intersection as shown on the cross-sectional sketch in Plate 19.

30. The Type 4 design stabilizer was used in conjunction with the Type 1 design bank protection at a 1V-on-2H slope as shown in Photo 6. The riprap on the bank remained in place as shown in Photo 9. A discharge of 3,000 cfs produced a maximum scour depth of 3.7 ft and an average scour depth of 2.9 ft (Plate 20). Increasing the peak discharge to 3,500 cfs after remolding the bed produced a maximum scour depth of 4.9 ft and an average of 4.5 ft (Plate 21).

31. The sheet piles were replaced with the concrete-capped stabilizers and the Type 2 design stabilizer protection was used with the Type 1 design bank protection for the concrete-capped stabilizers (Photo 10). Although the riprap bank remained intact with a discharge of 3,000 cfs, the higher discharge of 3,500 cfs undercut the toe of the bank protection. Displacement of the 1V-on-2H side slope protection followed the slope of the scour hole, filling in the scour and adding extra protection of the bed (Photos 11 and 12). Maximum scour depth at 3,000 cfs was 3.1 ft; average scour depth was 2.4 ft (Plate 22). At the design discharge, maximum scour increased to 3.4 ft and the average scour depth was 2.7 ft (Plate 23). The Type 1 design bank protection was considered adequate.

Concrete-Capped Stabilizer Design, 120-ft-Wide Channel

32. The section model was modified to reproduce a 40-ft-wide section of the 120-ft-wide channel portion of Mill Creek. The slope was increased to 0.016 ft/ft and seven concrete-capped stabilizers were spaced 60 ft apart. This represents typical channel conditions in the uppermost reaches of the Mill Creek flood-control channel.

33. The Type 2 design stabilizer protection was tested with discharges of 3,000 and 3,500 cfs. The differences between these tests and tests in the 70-ft-wide channel are decreases in unit discharges and stabilizer spacing and an increase in channel slope. Again the upstreammost and downstreammost sections were not used in determining average values, because they were not considered representative due to model boundary effects. Maximum scour depth was 4.1 ft and average scour depth was 3.8 ft with the 3,000-cfs discharge (Plate 24). The maximum scour depth increased to 5.6 ft and the average increased to

4.3 ft when the discharge was increased to 3,500 cfs (Plate 25). Photo 13 is a view, looking upstream, of the model with the Type 2 design stabilizer protection after passing 3,500 cfs. The material from upstream deposited on top of the downstream riprap, almost obscuring it from view in the lower sections of the model. Photo 14 is a closeup view of one of the sections immediately downstream of a stabilizer after passing the design discharge.

34. The unit discharge was increased to 50 cfs/ft which is equivalent to a discharge of 3,500 cfs in the 70-ft-wide channel section (Table 1). Maximum scour depth was 5.1 ft and average scour depth was 4.6 ft (Plate 26). In each test, initial transport of bed material was rapid but tapered off after the first hour. Most of the scour development occurred during the first 3 hr, with an increase in depth of 0.2 ft in the average depth during the next 16 hr. The riprap remained intact with a 3,500-cfs discharge.

35. There was no testing of sheet-pile stabilizers for this channel width because no sheet piles were to be placed in the 120-ft-wide channel section of the prototype.

Bank Toe Protection Design (Type 2)

36. An alternative (Type 2 design) bank protection scheme, consisting of a horizontal blanket of riprap (Figure 14), was tested in the 120-ft-wide channel section. Although the Type 1 design bank protection performed adequately in the 70-ft-wide channel section, a value engineering team in the Walla Walla District determined that a horizontal blanket would be less expensive to construct.

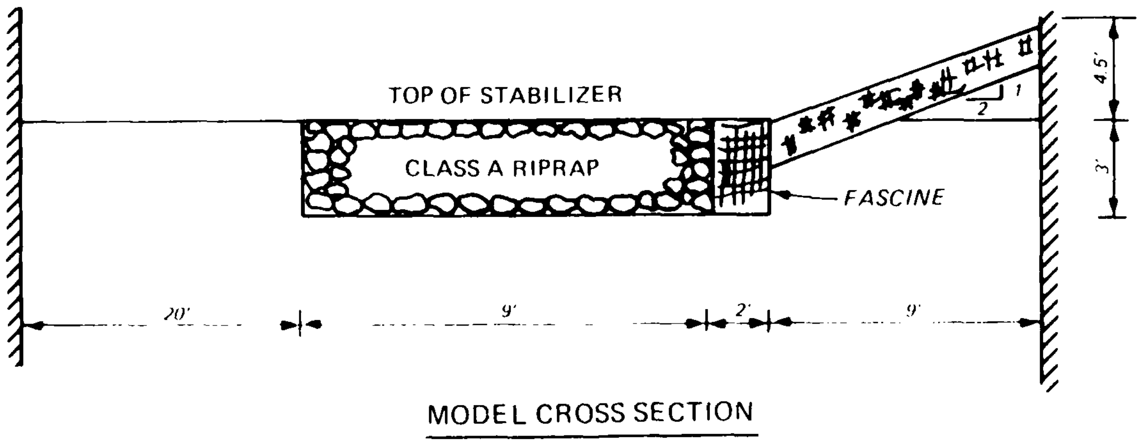

Figure 14. Type 2 bank toe protection

37. The Type 2 design stabilizer protection was used in conjunction with the Type 2 design bank protection. The hydraulic jump was forced away from the bank into the center of the channel causing the scour to be deepest in the center of the channel. Intermittent standing waves approximately 1 ft high were observed over the horizontal bank protection for discharges of 3,000 and 3,500 cfs, but no damage to the riprap occurred; the standing waves can be seen in Photo 15.

38. The tests showed that the maximum scour along the toe of the slope at a discharge of 3,000 cfs was 3.8 ft and the average scour depth was 3.0 ft (Plate 27). Increasing the discharge to 3,500 cfs produced a maximum scour depth of 3.9 ft and the average scour depth remained 3.0 ft (Plate 28). Photo 16 shows the total flume, looking upstream. The pattern of the sand in the photograph outlines the pattern of the flow during each test.

39. To determine if the Type 2 design bank protection would function with higher discharges, the unit discharge of 50 cfs/ft was run through the 120-ft-wide channel section. In this case, a maximum scour depth of 4.2 ft at an average of 3.6 ft were measured (Photo 17, Plate 29). Intermittent standing waves were also observed in this test, but the riprap remained intact (Photo 17). Due to the testing of a steeper channel slope, this test was considered adequate to recommend the Type 2 design bank protection for both the 70- and 120-ft-wide channel sections.

40. The overall recommended design determined from test results incorporates the Type 2 design stabilizer protection and Type 2 design bank protection for the concrete-capped stabilizers and the Type 4 design stabilizer protection and Type 2 design bank protection for the sheet piles in the 70-ft-wide channel section. The Type 2 design stabilizer protection and Type 2 design bank protection are recommended for the 120-ft-wide channel section.

Comparison of Model and Calculated Scour Depths

41. There is no widely recognized equation that can be used to predict scour depths downstream from low sills. Three equations have been proposed and are described by Simons and Senturk (1976). According to Simons and Senturk, the Schoklitsch (1932) equation "gives good results for relatively large D_{90} values." The equation can be written

$$D_{sm} + y = 3.75 \frac{H^{0.2} q^{0.5}}{D_{90}^{0.32}} \qquad (1)$$

where

D_{sm} = maximum depth of scour hole, ft

$D_{sm} + y$ = depth from deepest point of scour hole to downstream water-surface elevation, ft

H = head loss across sill, ft

q = unit discharge, ft^3/sec-ft

D_{90} = particle size for which 90 percent of the bed material is finer, mm

The Eggenberger (1943) equation was developed from laboratory flume data using fine sand beds and can be written

$$D_{sm} + y = 9.96 \frac{H^{0.5} q^{0.6}}{D_{90}^{0.4}} \qquad (2)$$

Mueller (1944) revised Eggenberger's equation using additional laboratory data with fine sand beds. His equation is

$$D_{sm} + y = 6.69 \frac{H^{0.5} q^{0.6}}{D_{90}^{0.4}} \qquad (3)$$

A fourth equation, used in the North Pacific Division, was developed by Veronese (1937) and reported by Scimemi (1947). This equation predicts scour depths downstream from weirs and can be written

$$D_{sm} + y = 1.32 H^{0.225} q^{0.54} \qquad (4)$$

Model data were used to evaluate predictive capabilities of the four equations for Mill Creek's gravel bed (Tables 3 and 4). Comparing scour depths predicted by the Schoklitsch (1932) and Veronese (1937) equations in Tables 3 and 4, it is immediately apparent that the equations do not conform to Froudian similitude laws and therefore have questionable value outside the range of data used in their development. The Eggenberger (1943) equation predicts scour depths considerably greater than those determined in the model studies.

The Mueller equation generally underestimates scour depths. It is concluded that model test results do not lend support to any of the proposed scour predictive equations.

42. Data collected during the verification phase of this study were used to develop an equation relating scour depth to unit discharge (Table 5, Figure 15). The dimensionless equation is

$$\frac{D_{sm}}{D_{90}} = 0.57\left(\frac{q}{g^{0.5}H^{1.5}}\right)^{1.33} \tag{5}$$

In this study D_{90}, H, and channel slope did not vary so that this equation cannot be recommended for use in other streams. Additional research work is required to develop an equation with widespread applicability.

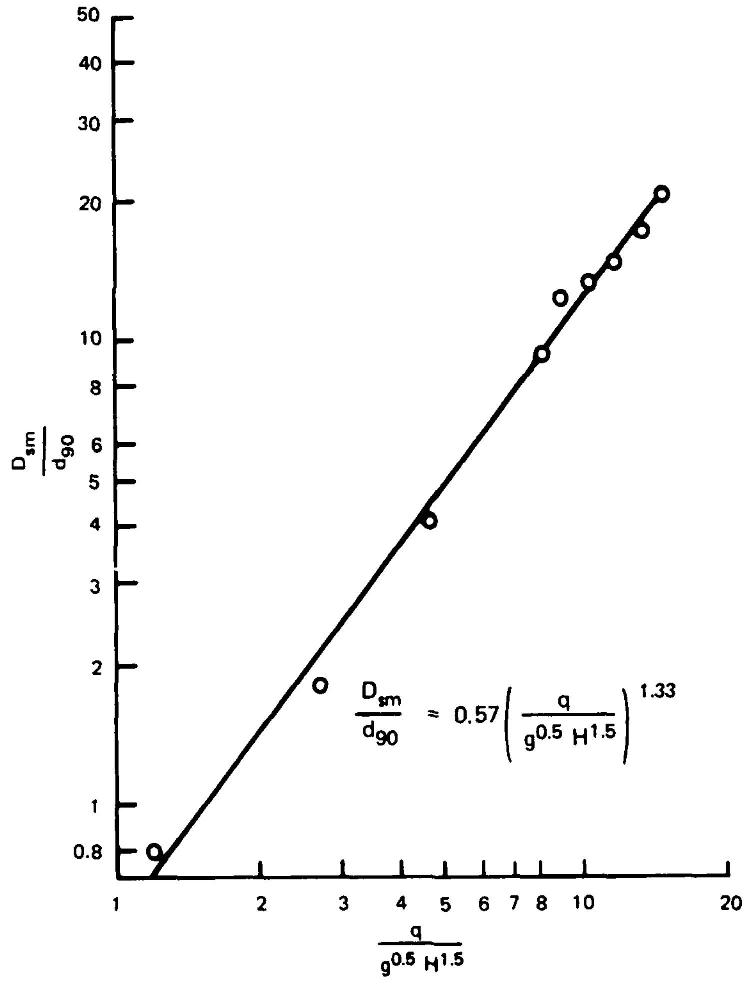

Figure 15. Relation between scour depth and unit discharge for Mill Creek channel

PART IV: CONCLUSIONS AND RECOMMENDATIONS

43. A physical model can be used to quantitatively design local scour protection in a gravel-bed stream where a sufficient portion of the bed material is coarse enough to be simulated according to the Froudian criteria. The physical model satisfactorily reproduced historical scour depths and armor layer gradations. In this study, the coarser 70 percent of the bed material was scaled appropriately. The finer material, when unprotected by an armor layer, was transported out of the model much as it would be in the natural stream.

44. Existing methods available for calculating scour depths downstream from low sills proved inadequate for Mill Creek. Equations proposed by Schoklitsch (1932), Eggenberger (1943), Mueller (1944), and Veronese (1937) were compared with model results. Schoklitsch's equation was found to apply to problems on the scale of laboratory flumes. The Eggenberger and Mueller equations predicted scour depths within ±100 percent of model values. The Veronese equation was not sensitive enough to changes in unit discharge. The relation $D_{sm}/D_{90} = 0.57\ (q/g^{0.5}H^{1.5})^{1.33}$ fits the data for Mill Creek reasonably well. Additional research is required to detemine if this type of equation is applicable for a wide range of head differentials and bed materials.

45. Concrete-capped stabilizers with sloping riprap aprons can be used successfully to stabilize Mill Creek channel for a unit discharge of 50 cfs/ft. The bed material of Mill Creek is composed of sand and gravel with a D_{50} of 22 mm and is generally armored with gravels. The maximum channel slope tested for Mill Creek was 0.016. Stabilizers are 60 to 70 ft apart. The apron developed as a result of the model study was composed of riprap with a W_{50} of 200 lb and on a slope of 1V on 3.1H. The top of the apron was located 2.5 ft below the top of the stabilizer. The apron's horizontal length was 8 ft. The banks were protected with the same size riprap placed on a 1V-on-2H slope. The top of the riprap was 2 ft above the top of the stabilizer. Toe protection was provided with a 9-ft-long and 3-ft-thick horizontal apron, the top of which was equal to the top of the stabilizers. With this recommended design, scour depths of about 5.3 ft can be expected in Mill Creek with the design discharge of 3,500 cfs.

46. Sheet-pile stabilizers with riprap protection upstream and downstream may also be used to stabilize Mill Creek channel for a unit discharge

of 50 cfs/ft. The maximum slope tested for this type of stabilizer was 0.013. Upstream protection consisted of riprap with a W_{50} of 60 lb, on a 1V-on-6H slope, extending for a horizontal distance of 5 ft. Locating the top of this riprap 0.5 ft below the top of the stabilizer increased the stability of the rock. The same downstream apron and side-slope protection used with the concrete-capped stabilizers was effective with the sheet-pile stabilizers. With this recommended design, scour depths of about 3.5 ft can be expected in Mill Creek with a design discharge of 3,500 cfs.

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Table 1
Conversion of Discharges to Unit Discharges

Discharge, cfs	Channel Width, ft	Unit Discharge, cfs/ft
3,000	70	42.86
3,500	70	50
4,000	70	57.14
4,500	70	64.29
5,000	70	71.43
3,000	120	25
3,500	120	29.17

Table 2
Riprap Gradations for Mill Creek Channel

Weight, lb		Percentage
	Class A Riprap	
1,044 - 417		25
309 - 209		50
155 - 65		25
	Class B Riprap	
309 - 124		50
92 - 62		50
46 - 19		--

Table 3

Comparison of Measured and Calculated Scour Depths

Simulated Discharge	Measured Data From Model					Calculated Scour Depth			
						Schoklitsch	Eggenberger	Mueller	Veronese
	q	H	y	D_{90}	D_{sm}	D_{sm}	D_{sm}	D_{sm}	D_{sm}
cfs	cfs/ft	ft	ft	mm	ft	ft	ft	ft	ft
3,000	1.36	0.09	0.40	12.0	0.48	0.82	0.93	0.49	0.37
3,500	1.58	0.09	0.49	12.0	0.52	0.82	0.96	0.49	0.49
4,000	1.81	0.09	0.53	12.0	0.58	0.88	1.04	0.53	0.53
4,500	2.03	0.09	0.58	12.0	0.67	0.91	1.11	0.56	0.54
5,000	2.26	0.09	0.66	12.0	0.82	0.91	1.14	0.55	0.53

Table 4

Comparison of Simulated* and Calculated Scour Depths

Simulated Discharge	Measured Data From Model					Calculated Scour Depth			
						Schoklitsch	Eggenberger	Mueller	Veronese
	q	H	y	D_{90}	D_{sm}	D_{sm}	D_{sm}	D_{sm}	D_{sm}
cfs	cfs/ft	ft	ft	mm	ft	ft	ft	ft	ft
3,000	42.9	0.9	4.0	120	4.8	1.2	9.3	4.9	5.8
3,500	50.0	0.9	4.9	120	5.2	0.71	9.6	4.9	5.8
4,000	57.1	0.9	5.3	120	5.8	0.70	10.4	5.3	6.2
4,500	64.3	0.9	5.8	120	6.7	0.56	11.1	5.6	6.4
5,000	71.4	0.9	6.6	120	8.2	0.10	11.4	5.5	6.3

* Model measurement converted to prototype scale based on laws of similitude.

Table 5

Relationship Between Scour Depth and Unit Discharge

Unit Discharge q cfs/ft	Head Loss Between Stabilizers H, ft	D_{90} ft	Scour Below Grade D_{sm} ft	$\frac{D_{sm}}{D_{90}}$	$\frac{q}{g^{0.5}H^{1.5}}$
5.7	0.9	0.39	0.3	0.8	1.2
12.9	0.9	0.39	0.5	1.8	2.7
22.9	0.9	0.39	1.6	4.1	4.7
39.4	0.9	0.39	3.6	9.3	8.1
42.9	0.9	0.39	4.8	12.3	8.9
50.0	0.9	0.39	5.2	13.3	10.3
57.1	0.9	0.39	5.8	14.9	11.8
64.3	0.9	0.39	6.7	17.2	13.3
71.4	0.9	0.39	8.2	20.8	14.8

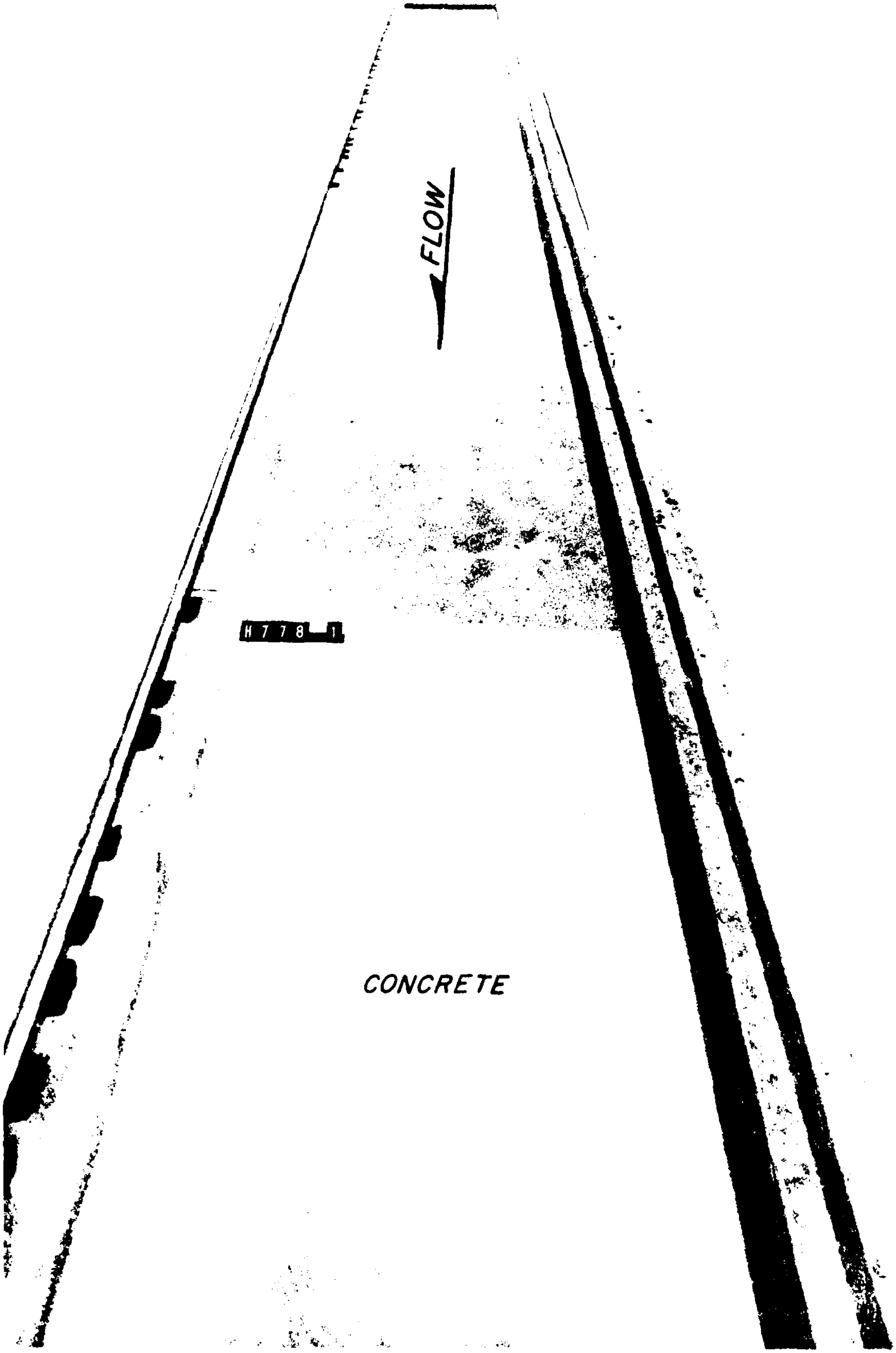


Photo 1. 1:10-scale model, 70-ft simulated width

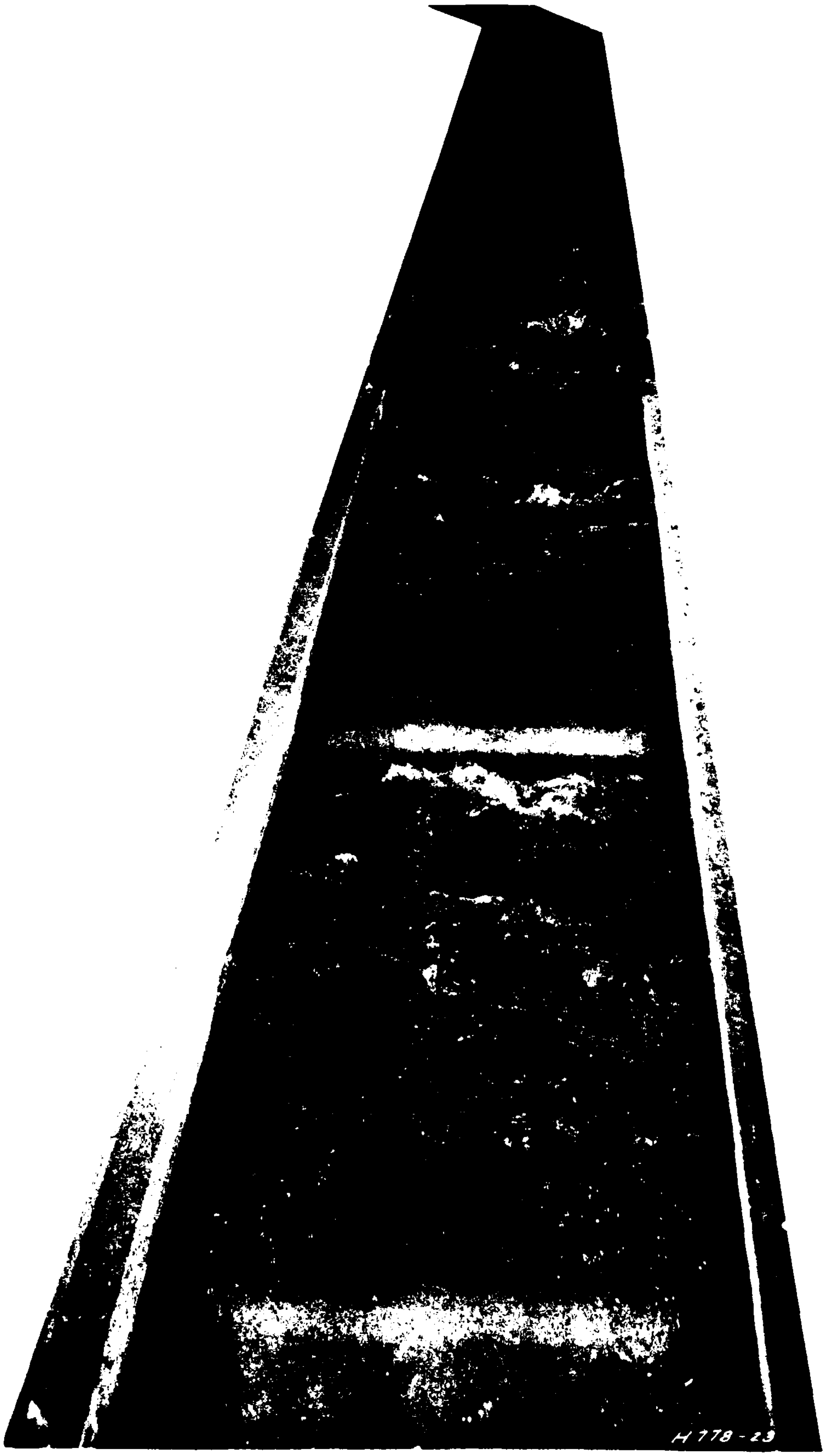


Photo 2. Mill Creek channel, Q = 3,000 cfs

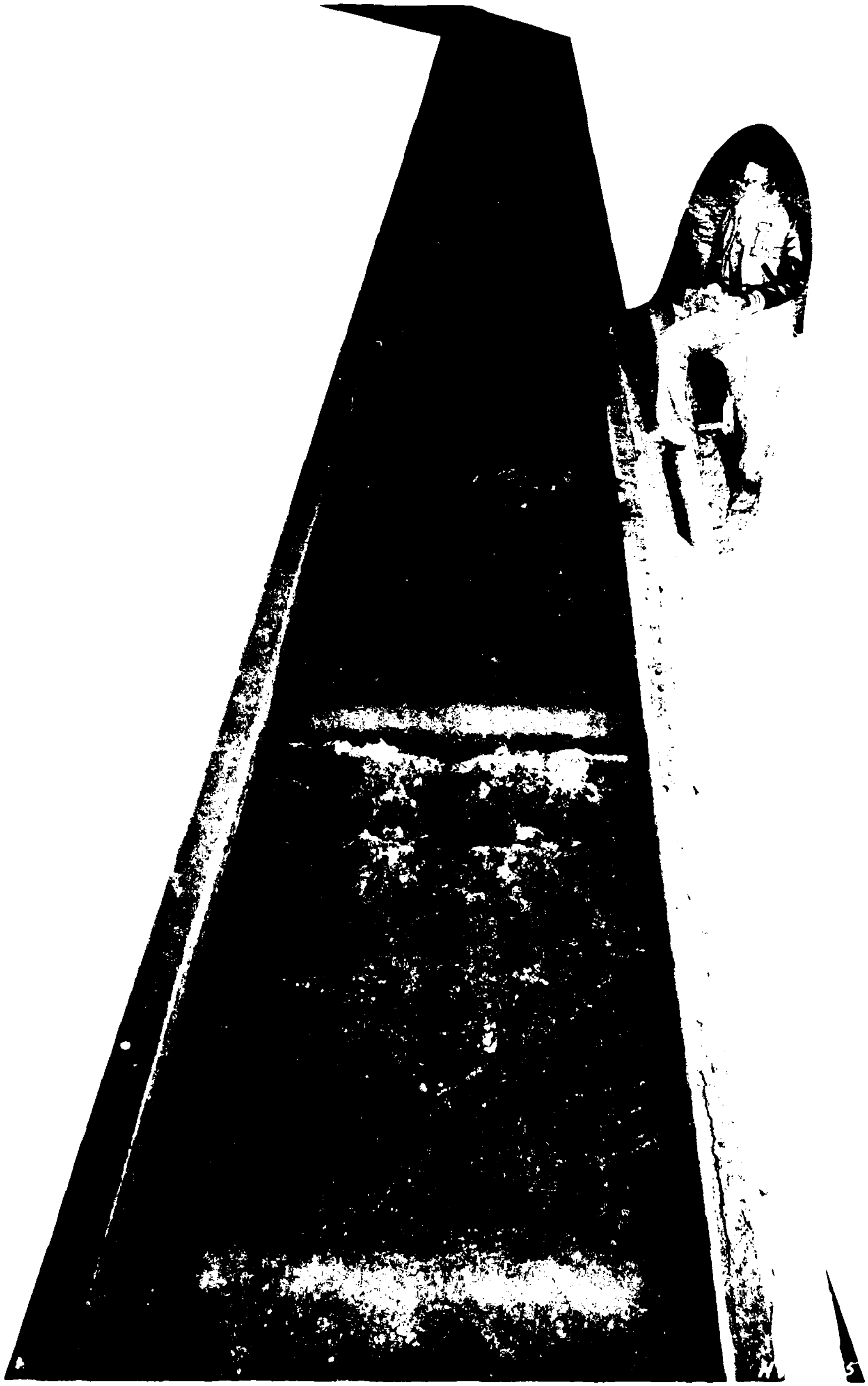


Photo 3. Mill Creek channel, Q = 5,000 cfs

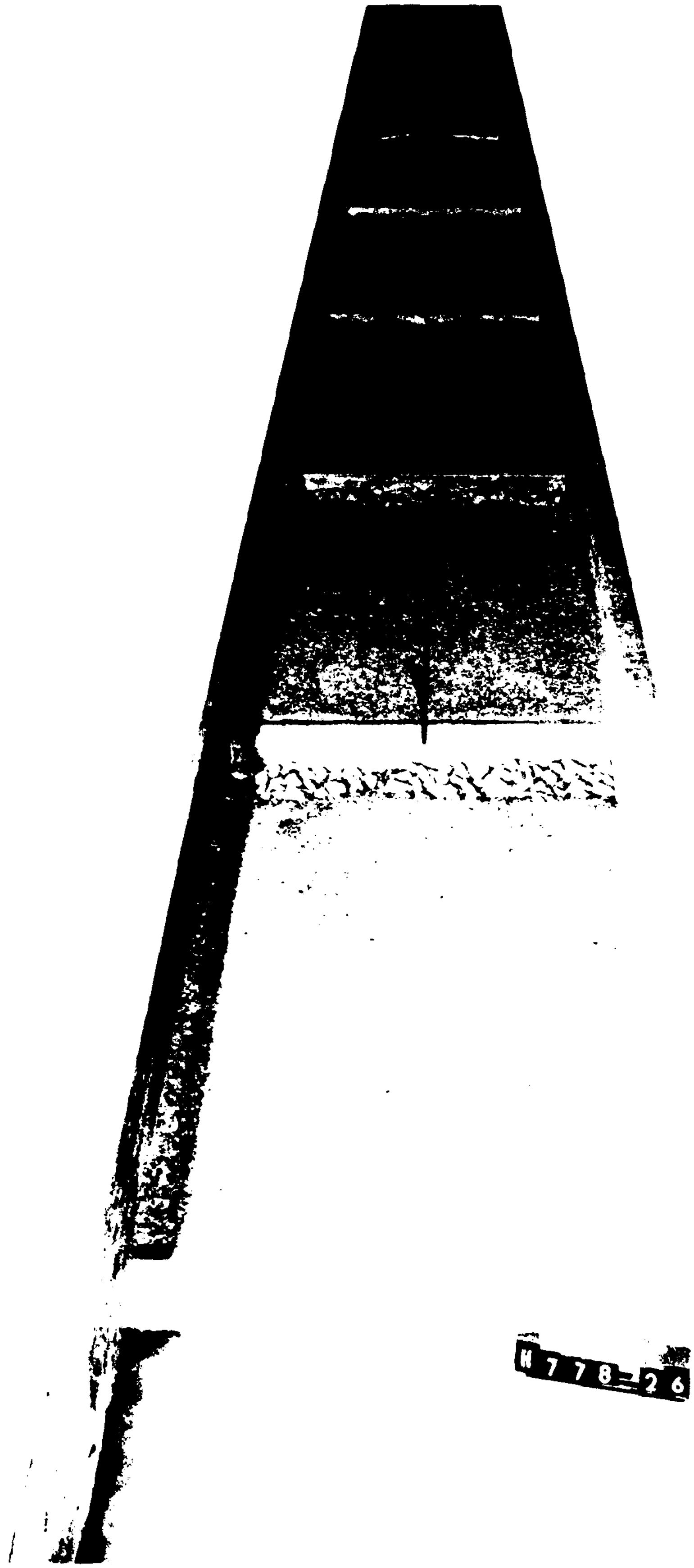


Photo 4. Type 1 (original) design stabilizer protection,
70-ft simulated width

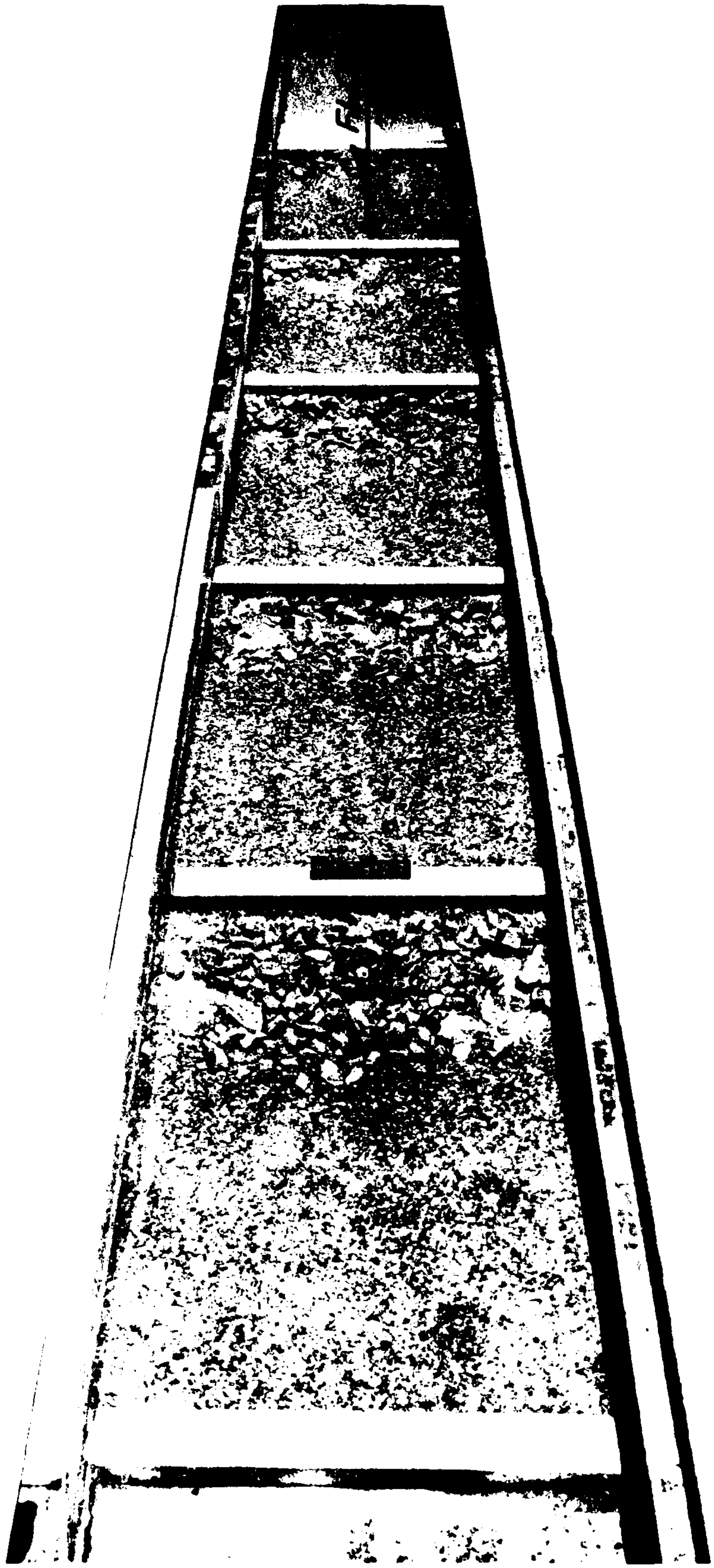


Photo 5. Type 1 (original) design stabilizer protection, Q_{max} = 4,000 cfs, 70-ft simulated width

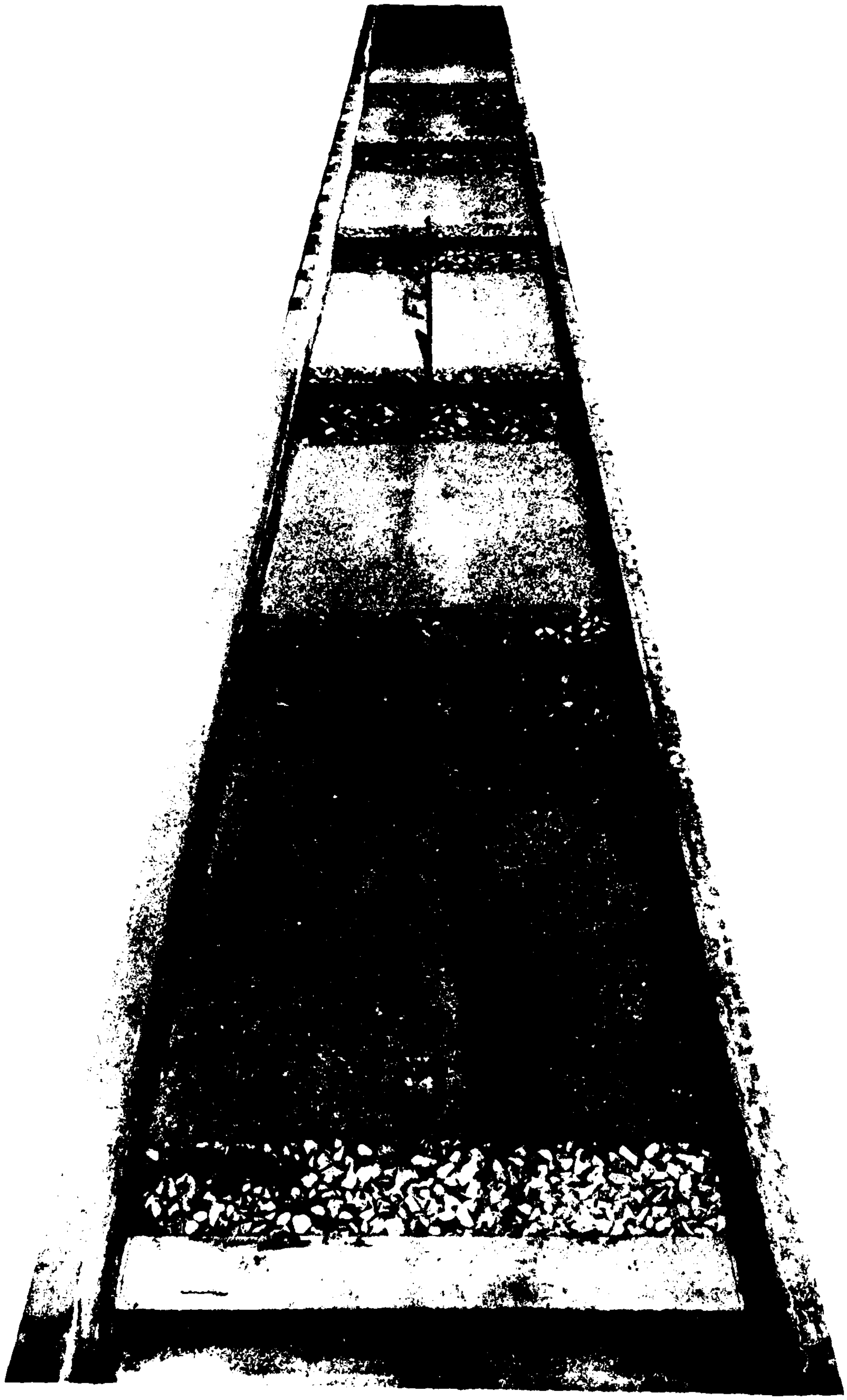


Photo 6. Type 3 design stabilizer protection,
70-ft simulated width

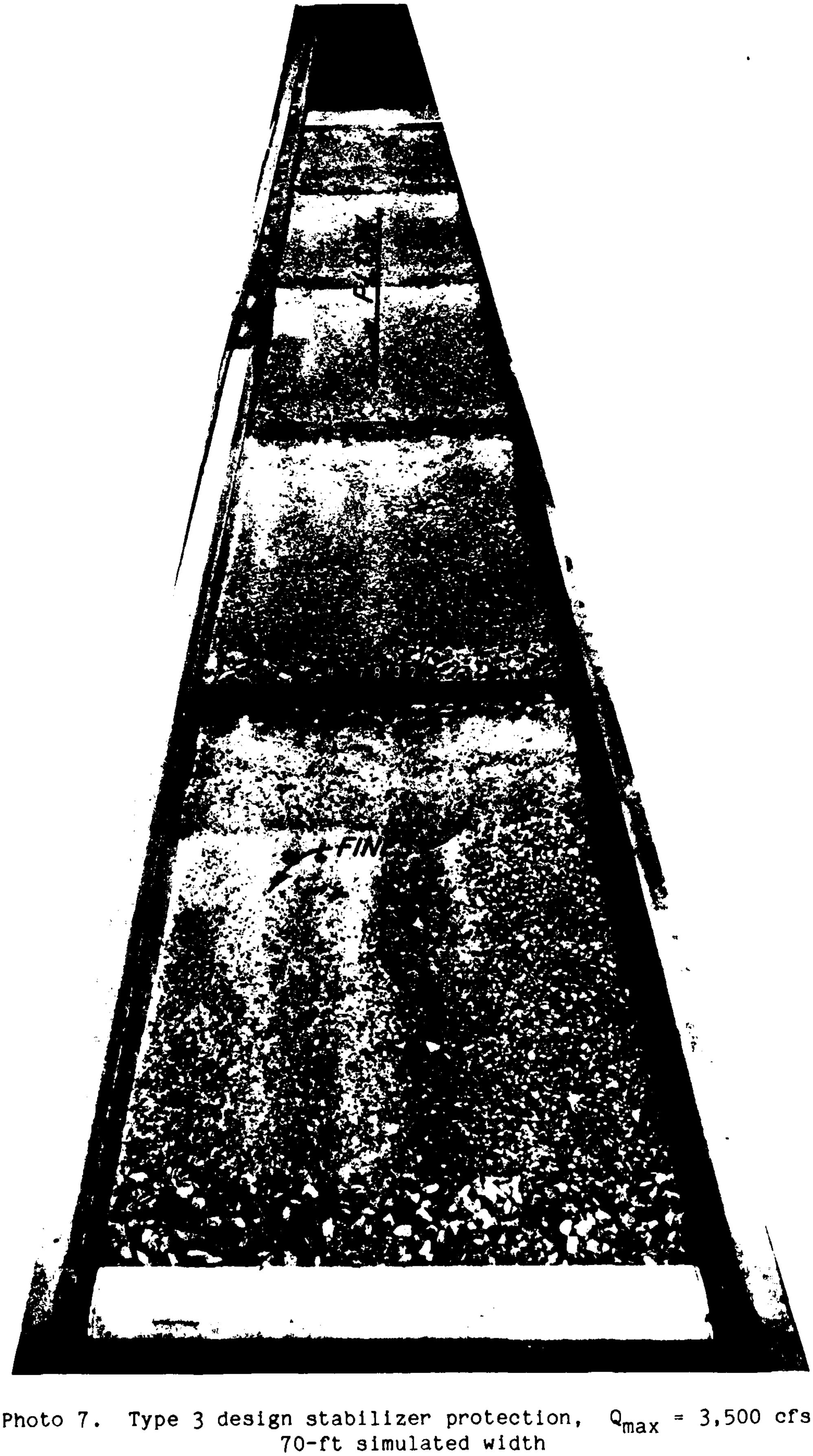


Photo 7. Type 3 design stabilizer protection, Q_{max} = 3,500 cfs, 70-ft simulated width

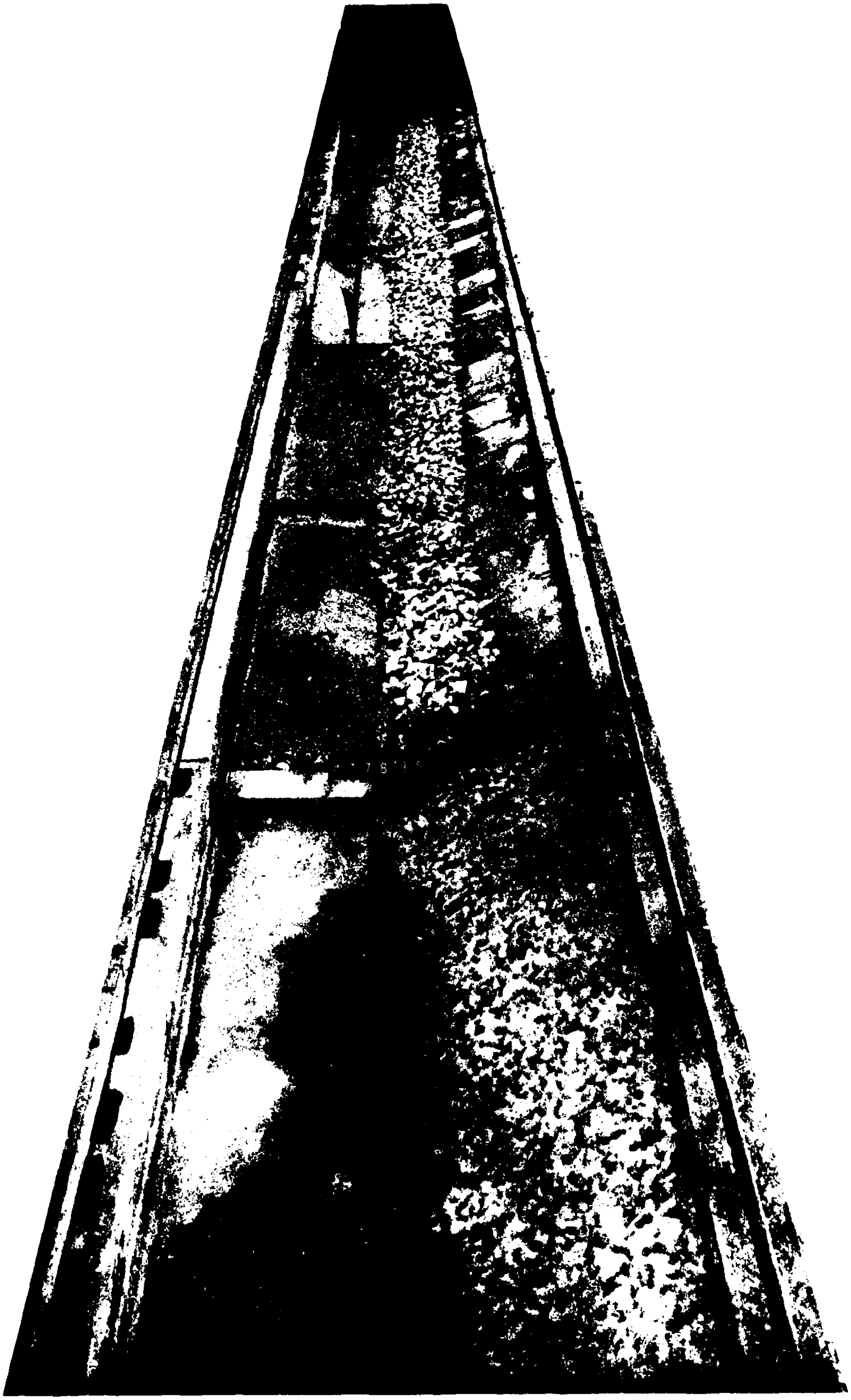


Photo 8. Type 4 design stabilizer protection and Type 1 design bank protection; 70-ft simulated width

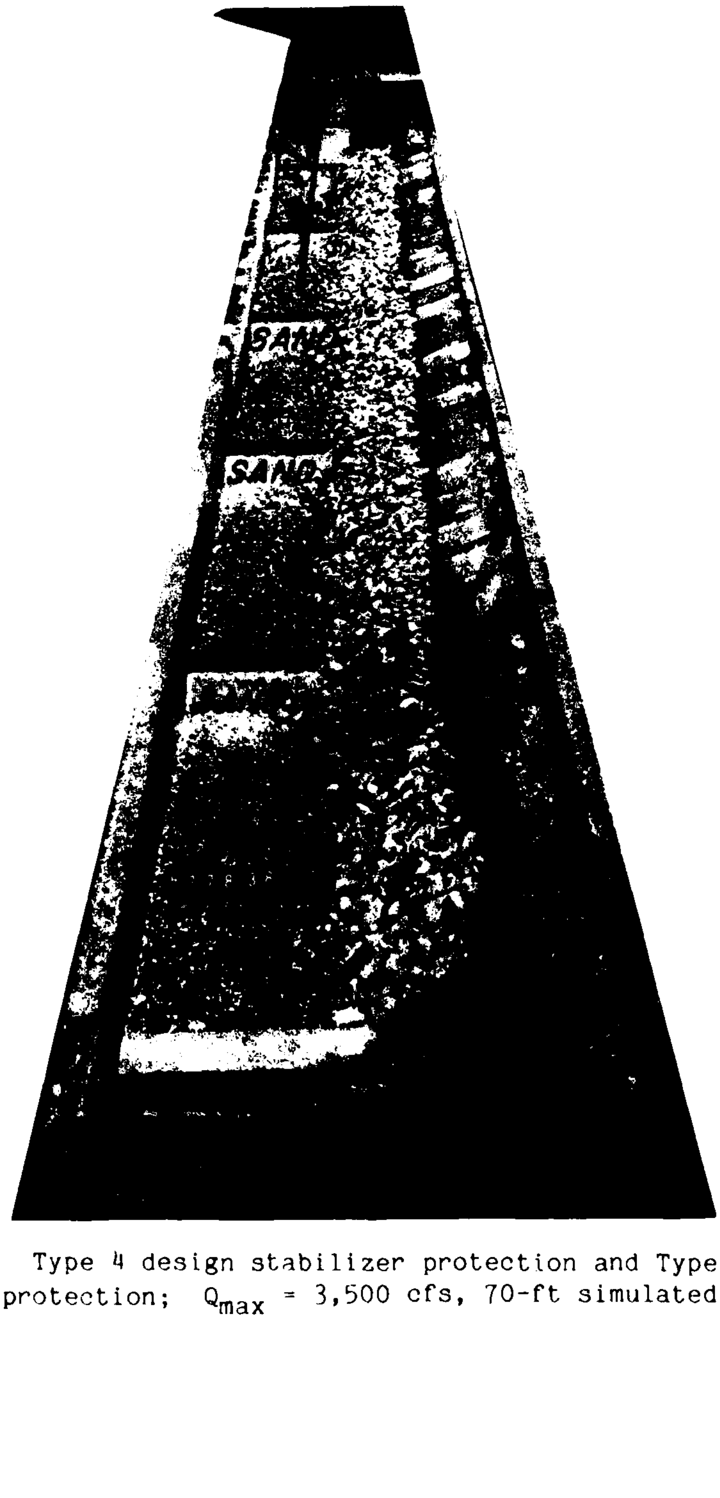


Photo 9. Type 4 design stabilizer protection and Type 1 design bank protection; Q_{max} = 3,500 cfs, 70-ft simulated width

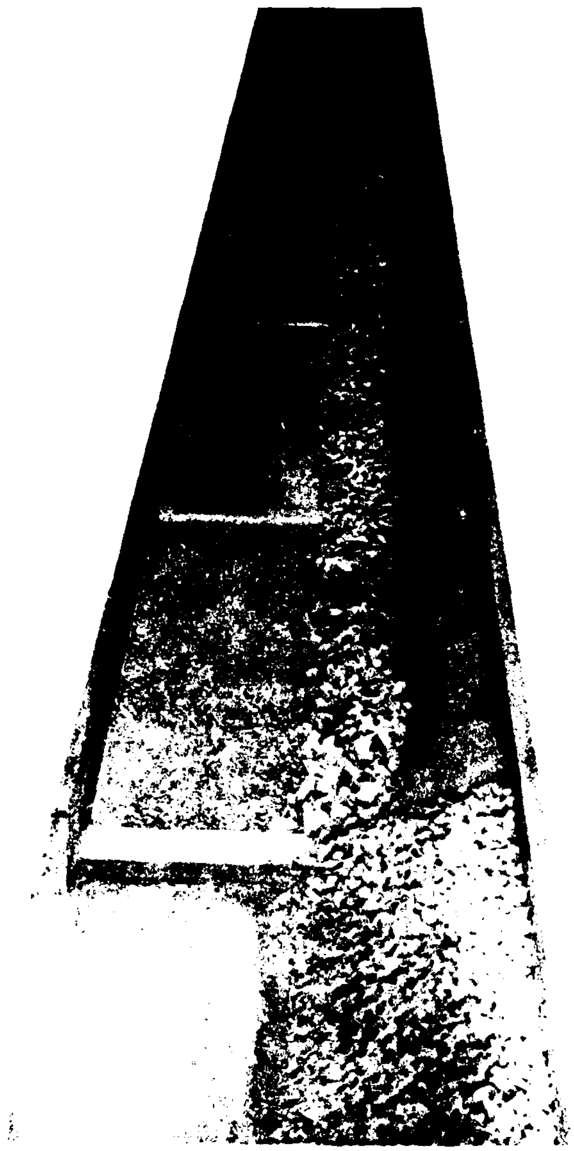


Photo 10. Type 2 design stabilizer protection and Type 1 design bank protection; 70-ft simulated width

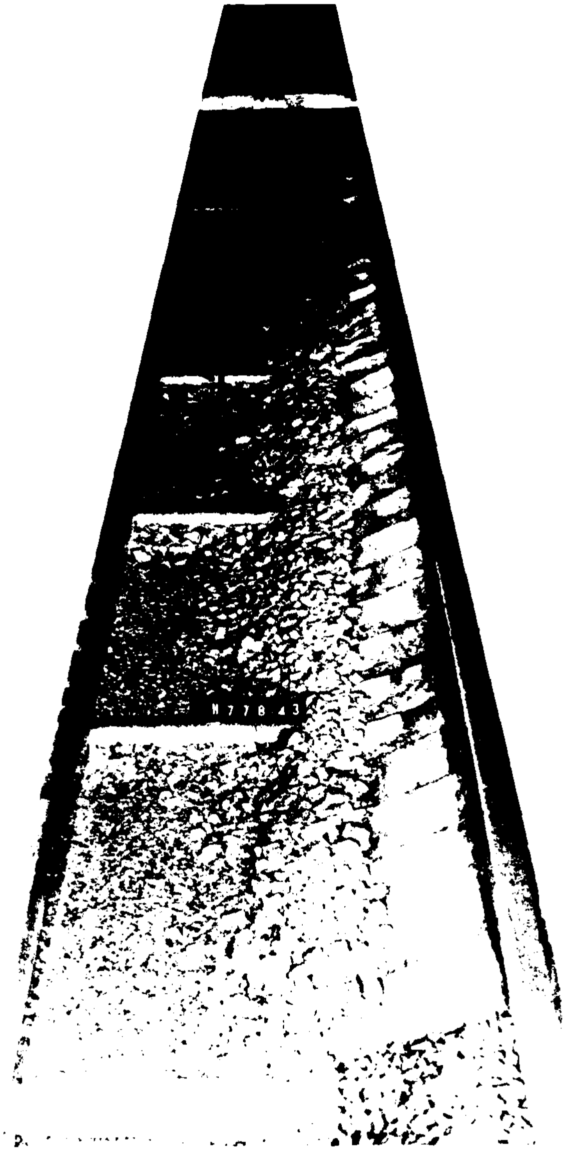


Photo 11. Type 2 design stabilizer protection and Type 1 design bank protection; Q_{max} = 3,500 cfs, 70-ft simulated width

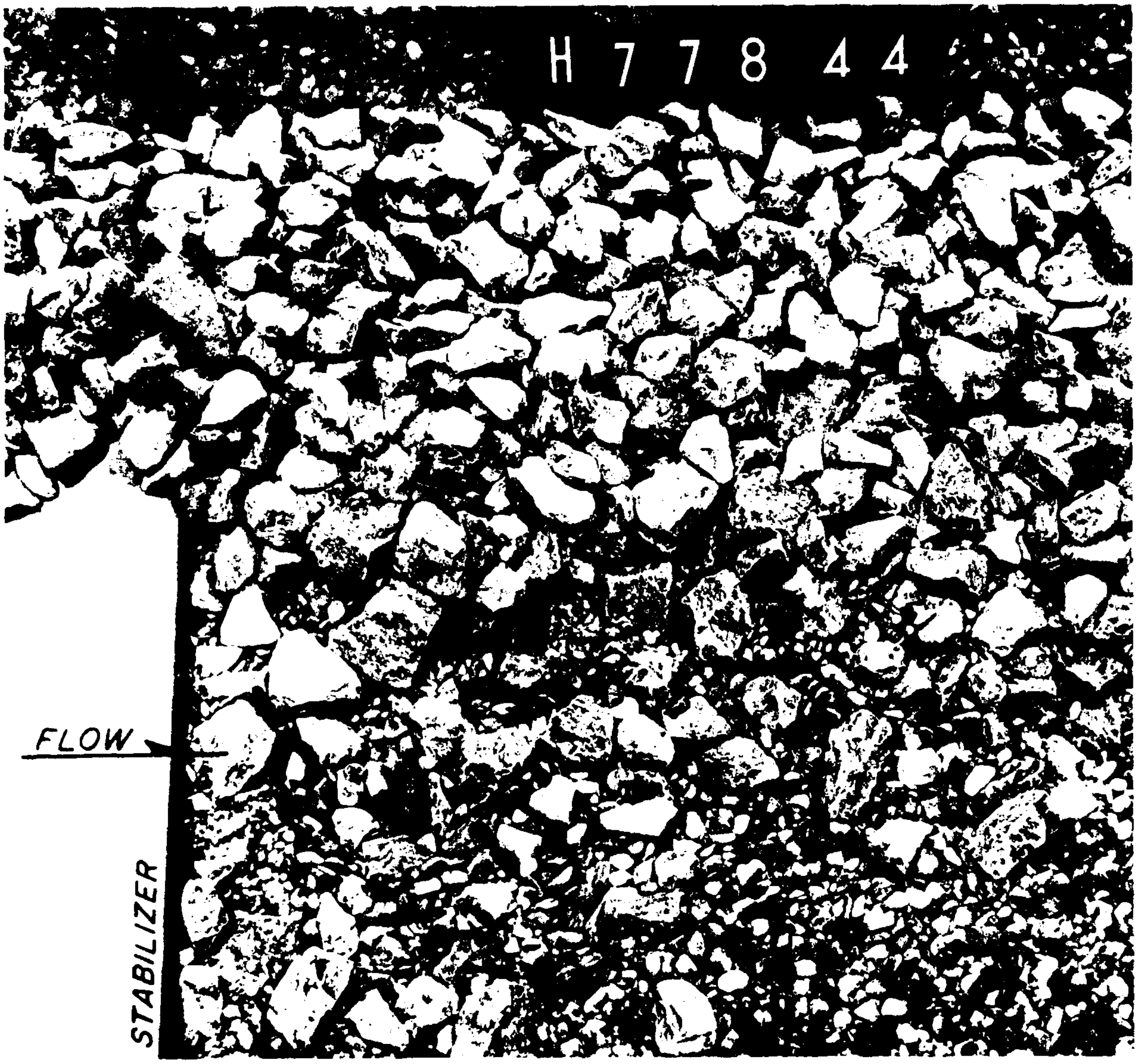


Photo 12. Closeup of Type 2 design stabilizer of protection and Type 2 design bank protection, Q_{max} = 3,500 cfs, 70-ft simulated width

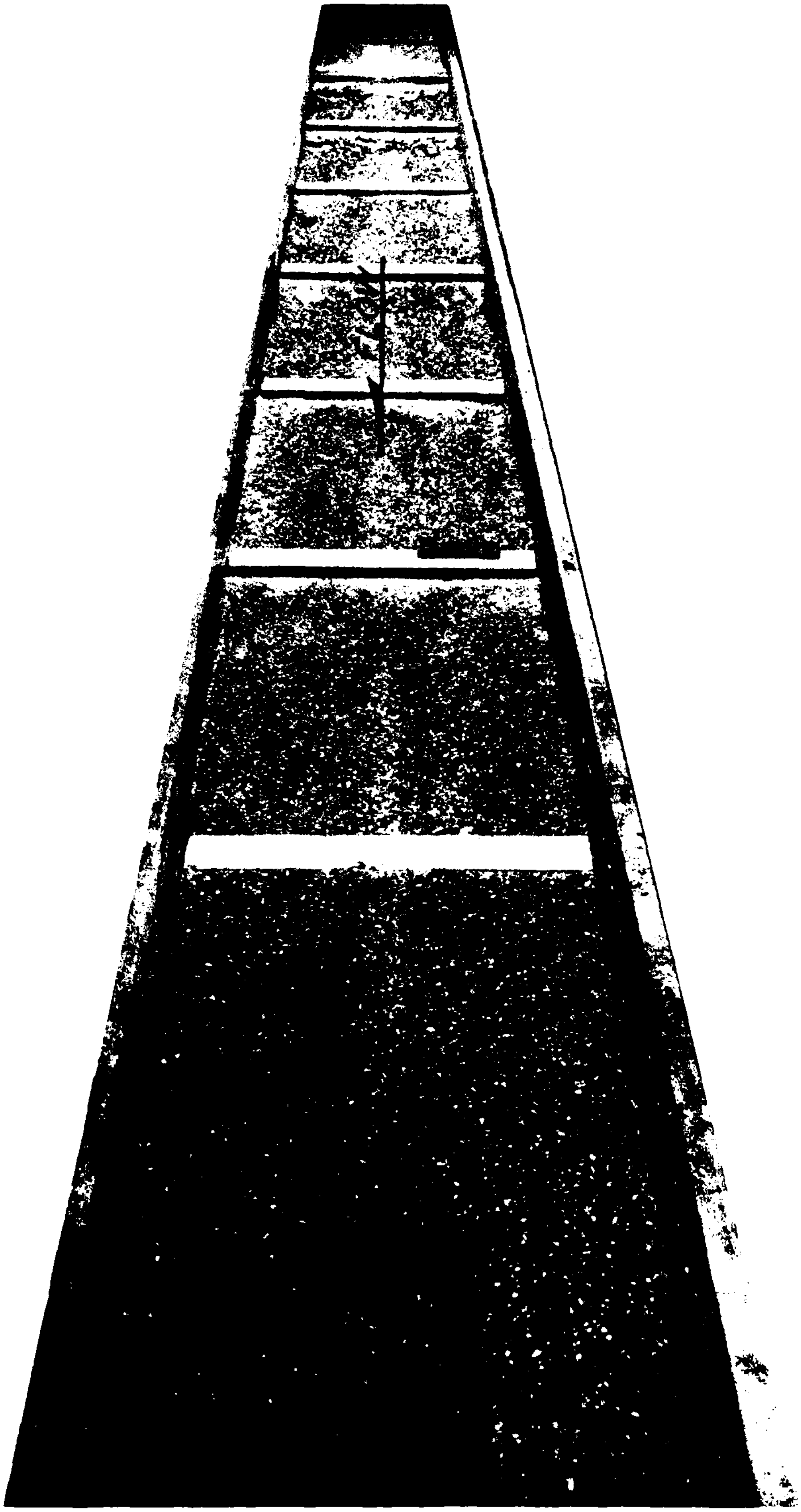


Photo 13. Type 2 design stabilizer protection, Q_{max} = 3,500 cfs, 120-ft simulated width

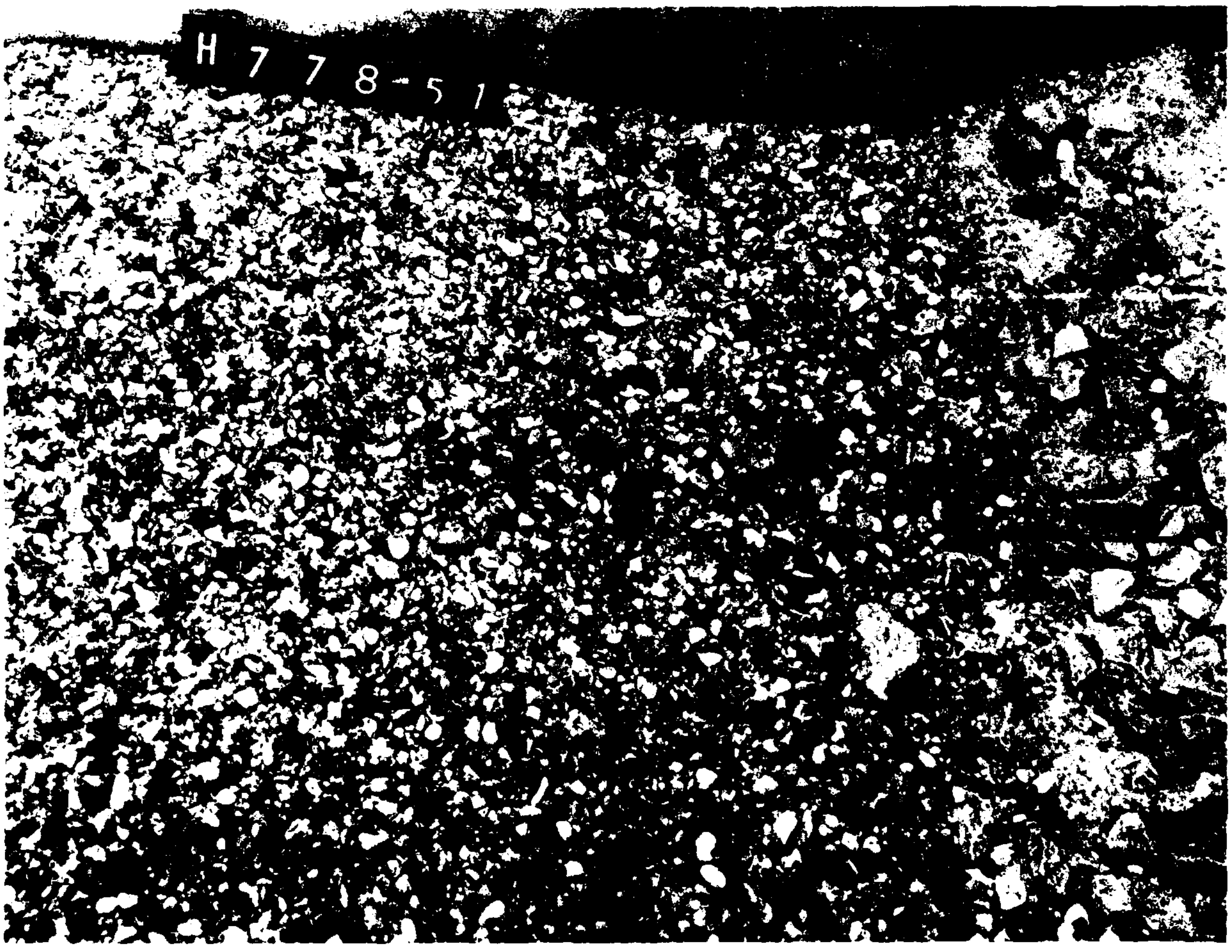


Photo 14. Closeup of Type 2 design stabilizer protection, Q_{max} = 3,500 cfs , 120-ft simulated width

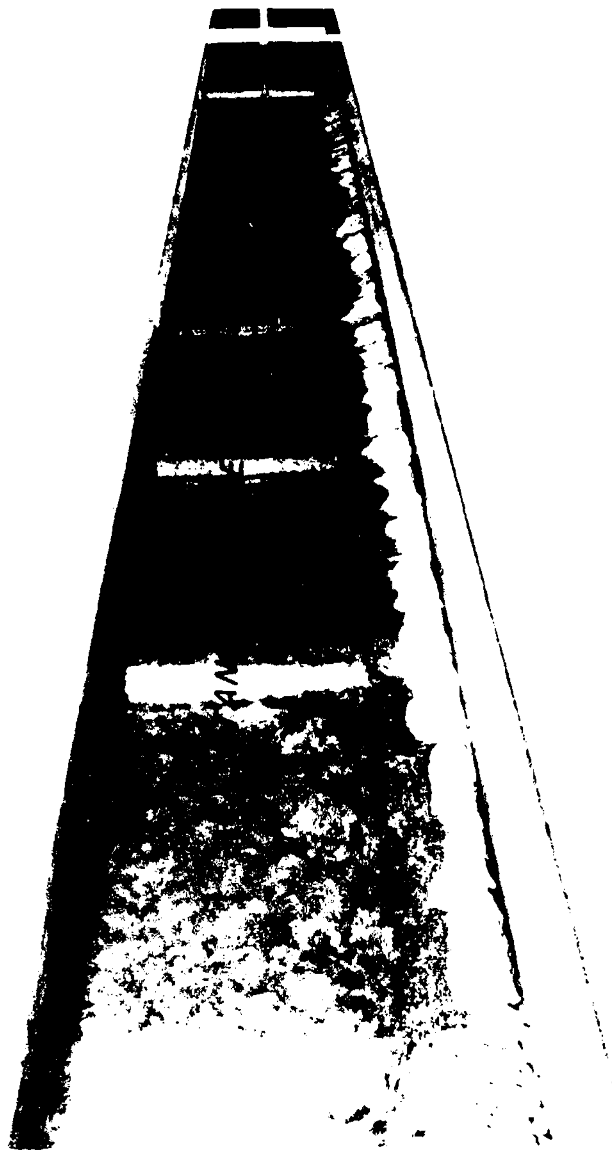


Photo 15. Type 2 design stabilizer protection and Type 2 design bank protection; Q_{max} = 3,500 cfs, 120-ft simulated width

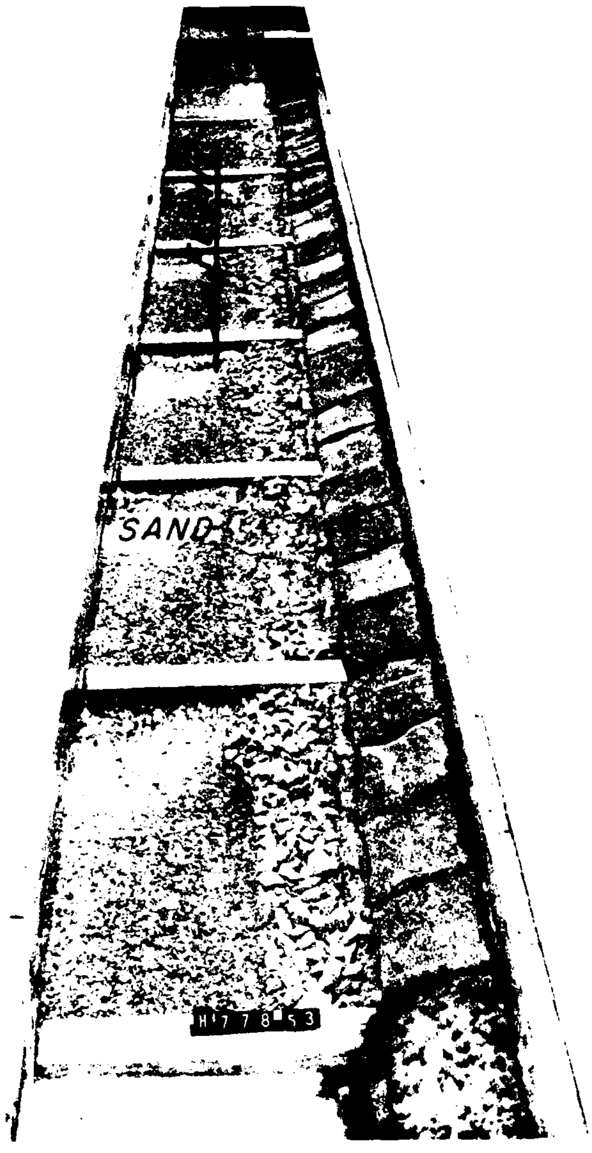


Photo 16. Type 2 design stabilizer protection and Type 2 design bank protection; Q_{max} = 3,500 cfs, 120-ft simulated width

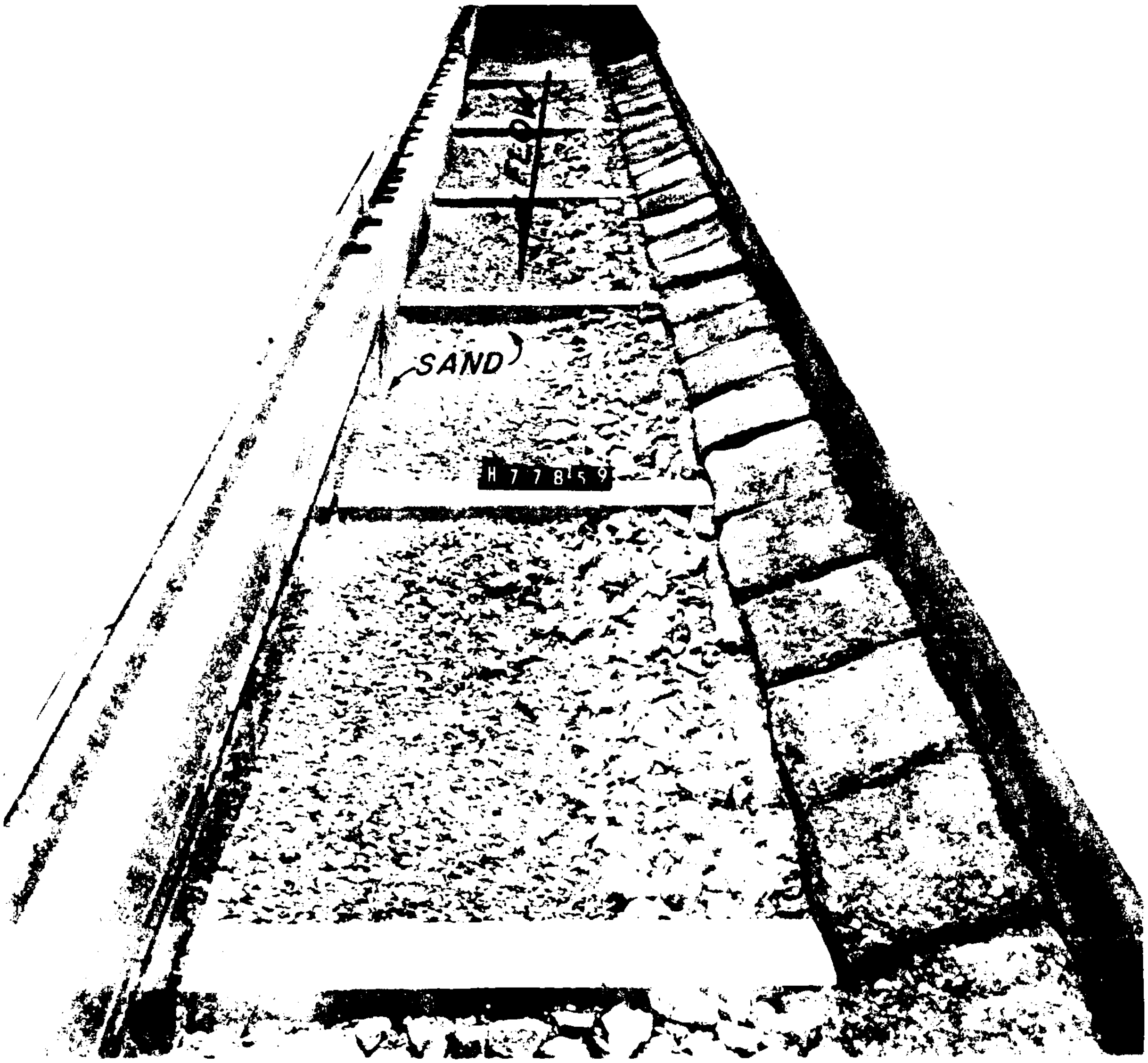
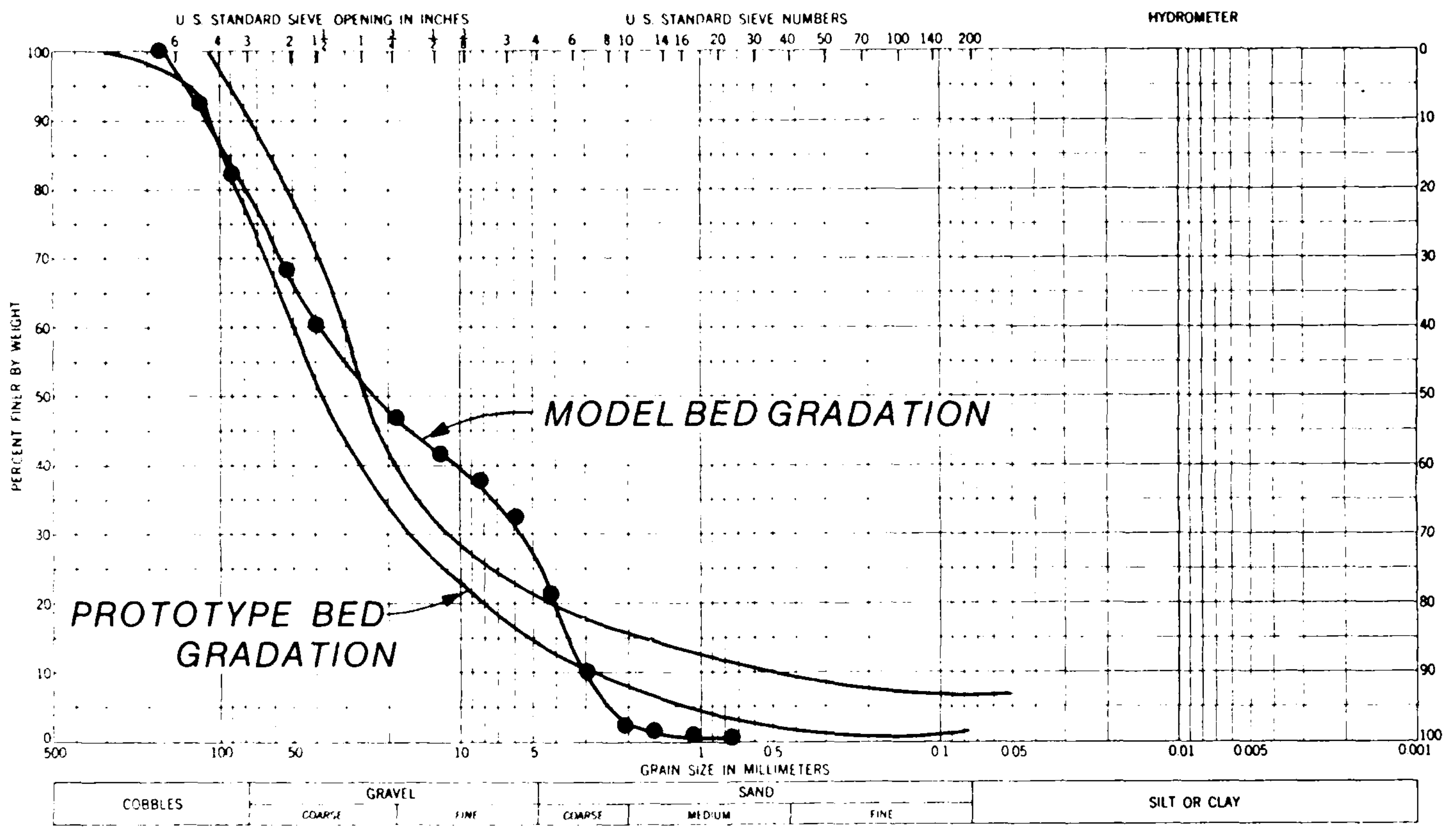
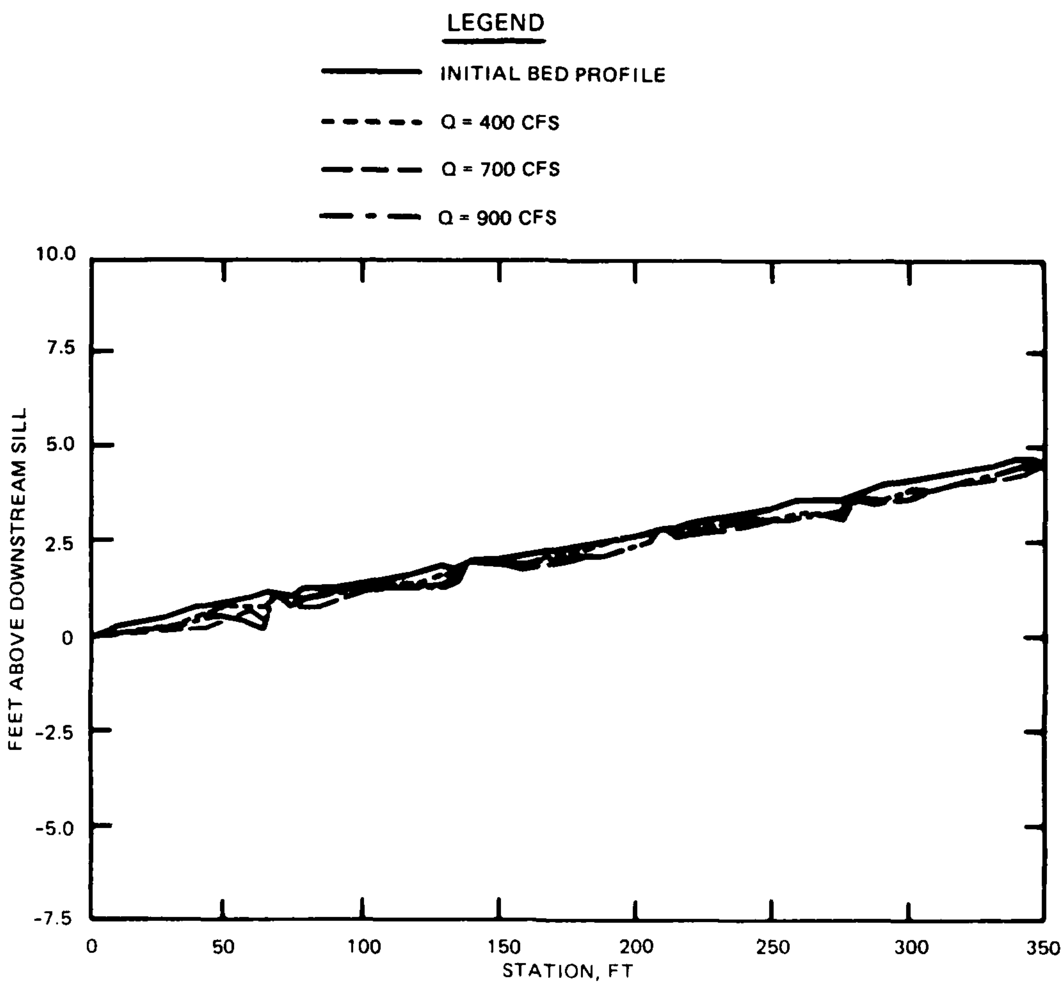


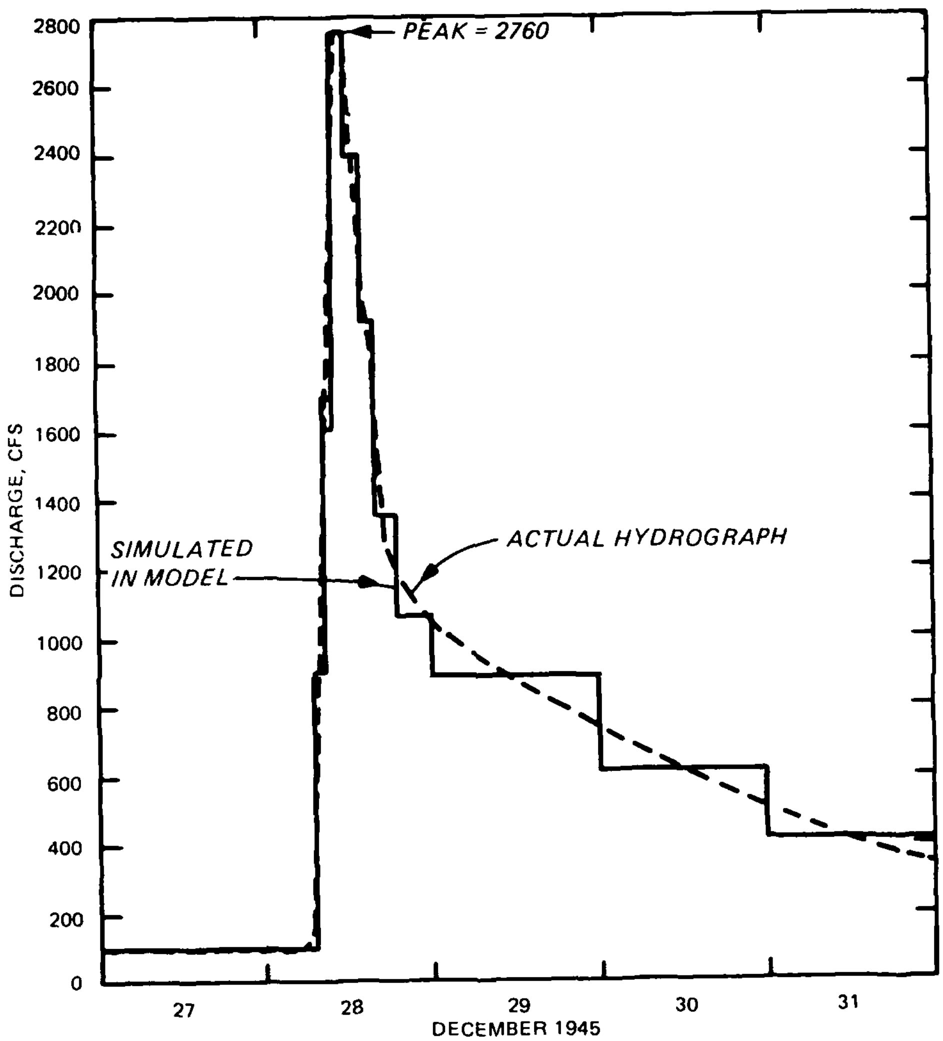
Photo 17. Type 2 design stabilizer protection and Type 2 design bank protection; Q_{max} = 50 cfs/ft, 120-ft simulated width



MODEL AND PROTOTYPE
BED GRADATIONS

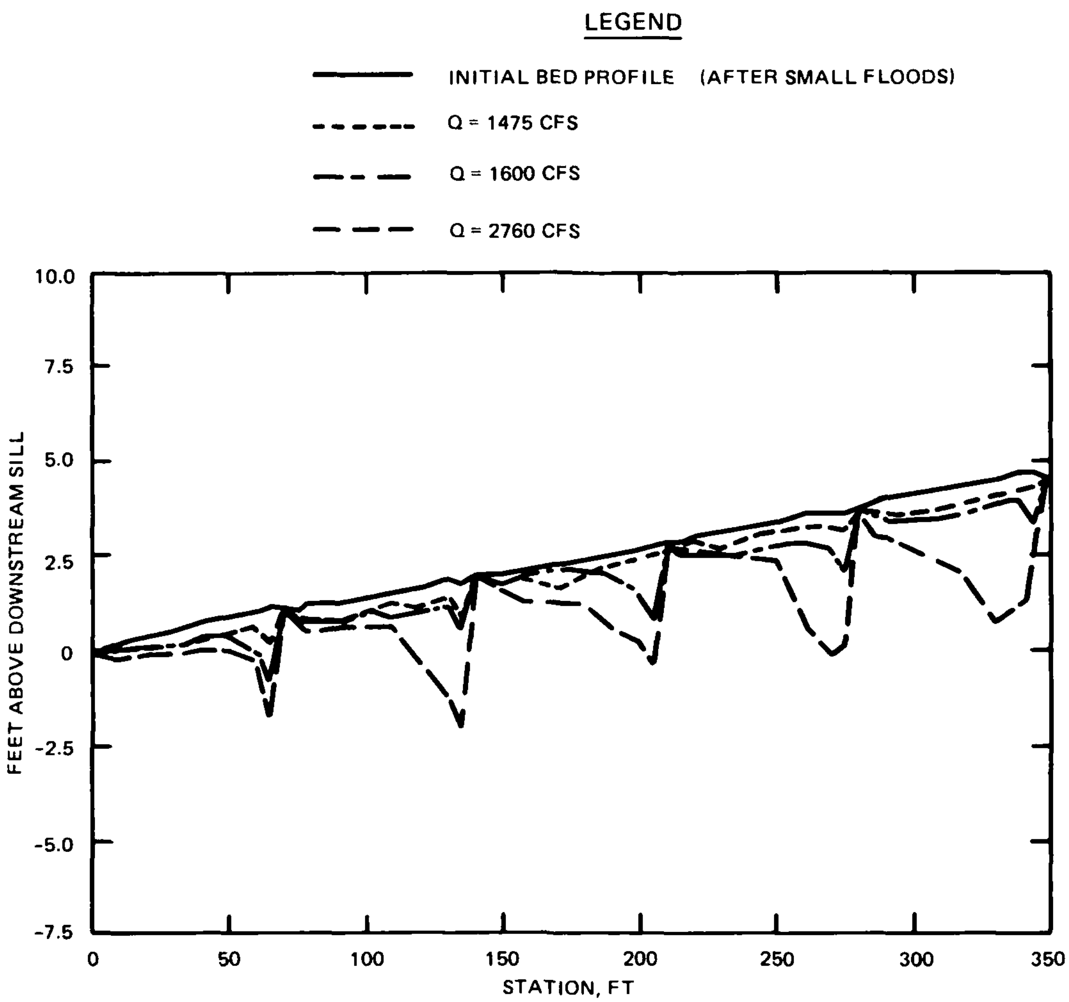


BED PROFILES
SMALL FLOODS



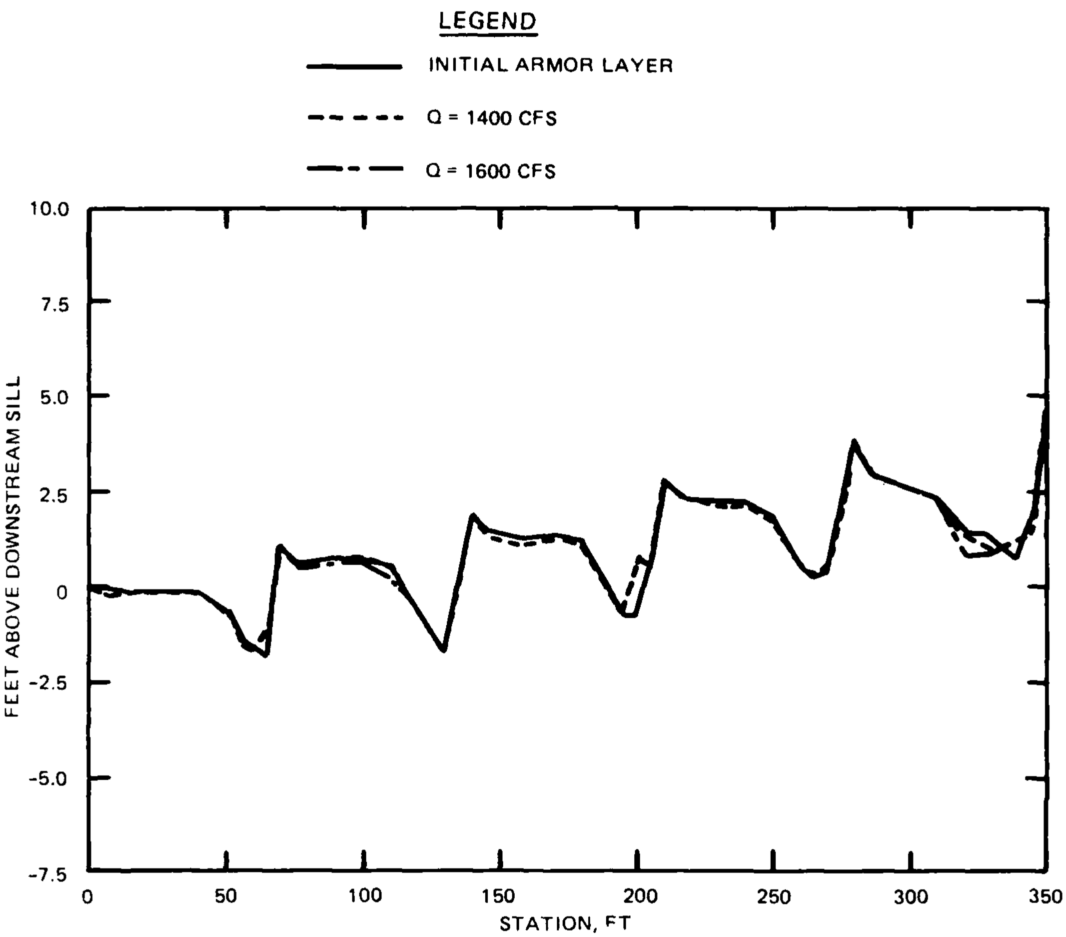
1945 FLOOD
HYDROGRAPH

PLATE 3

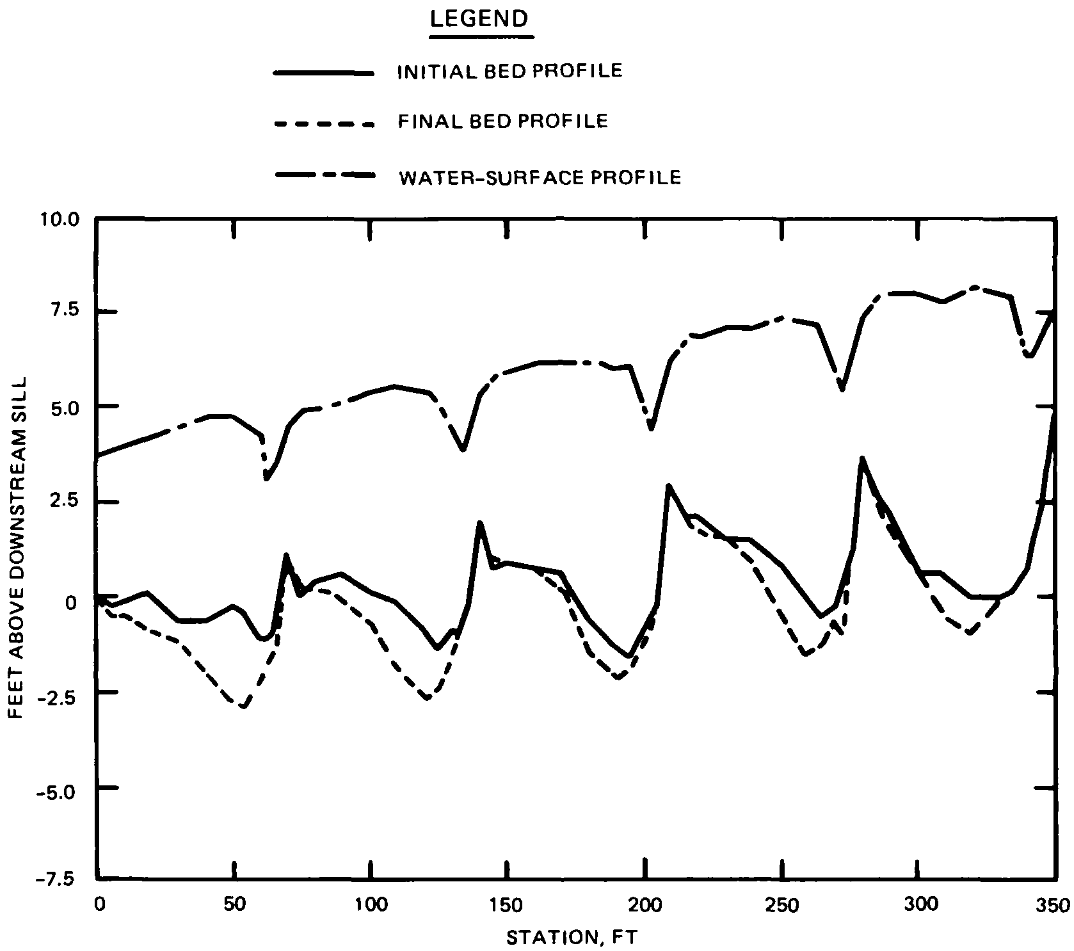


BED PROFILES
1945 FLOOD HYDROGRAPH

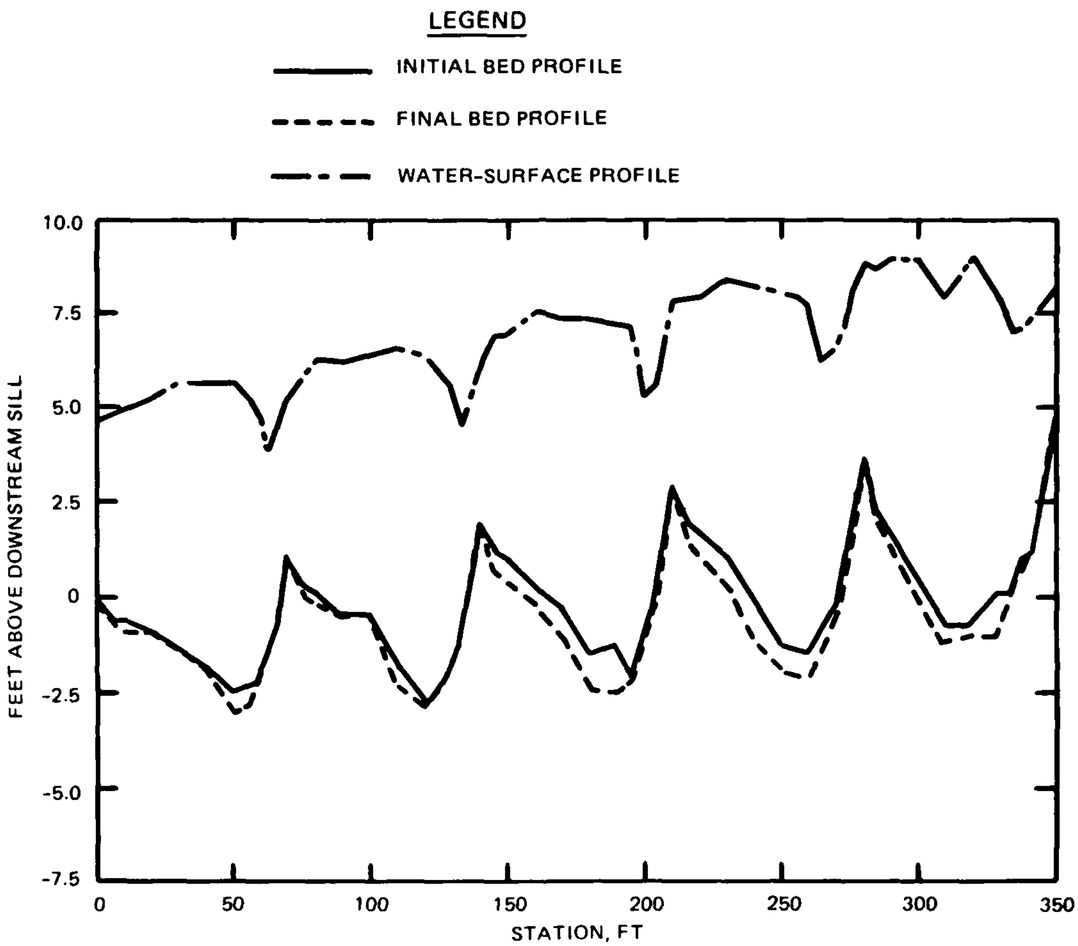
PLATE 4



BED PROFILES
1946 FLOOD HYDROGRAPH



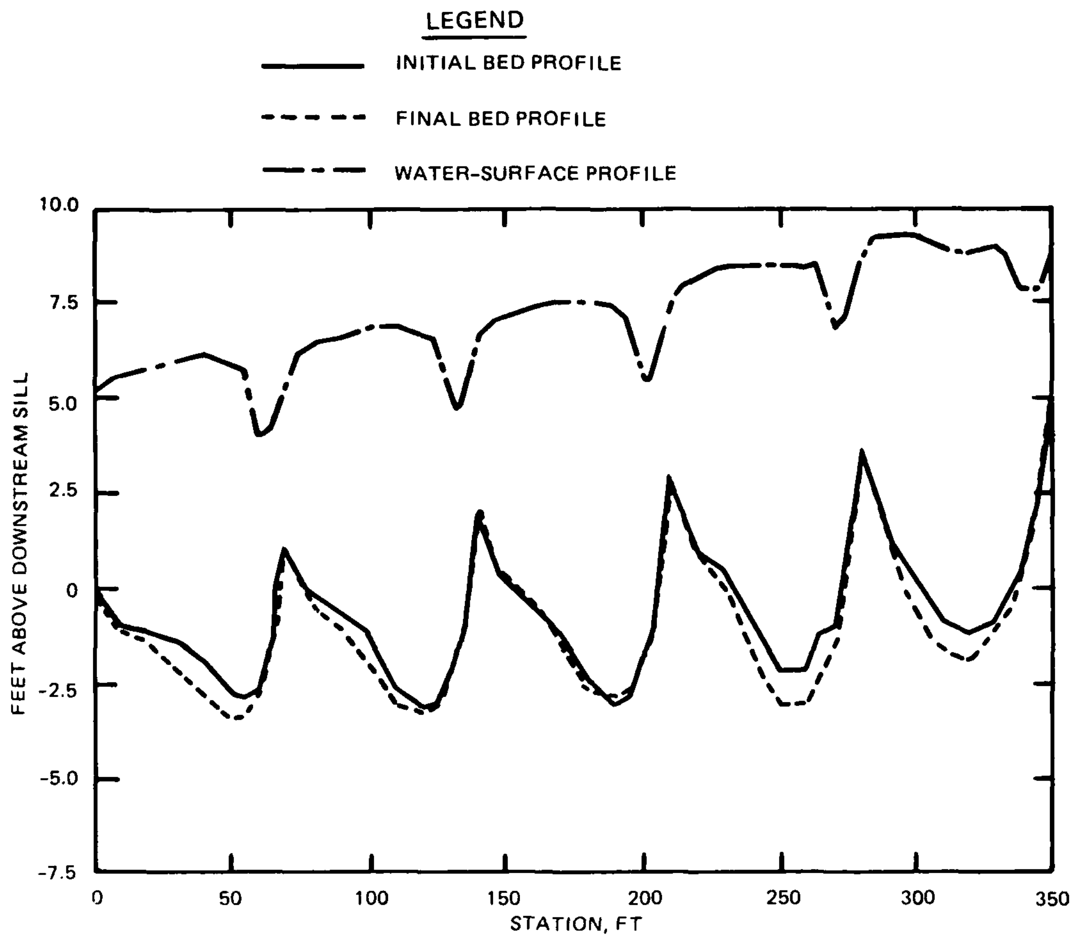
BED AND WATER-SURFACE PROFILES
UNPROTECTED CHANNEL
DISCHARGE 3,000 CFS



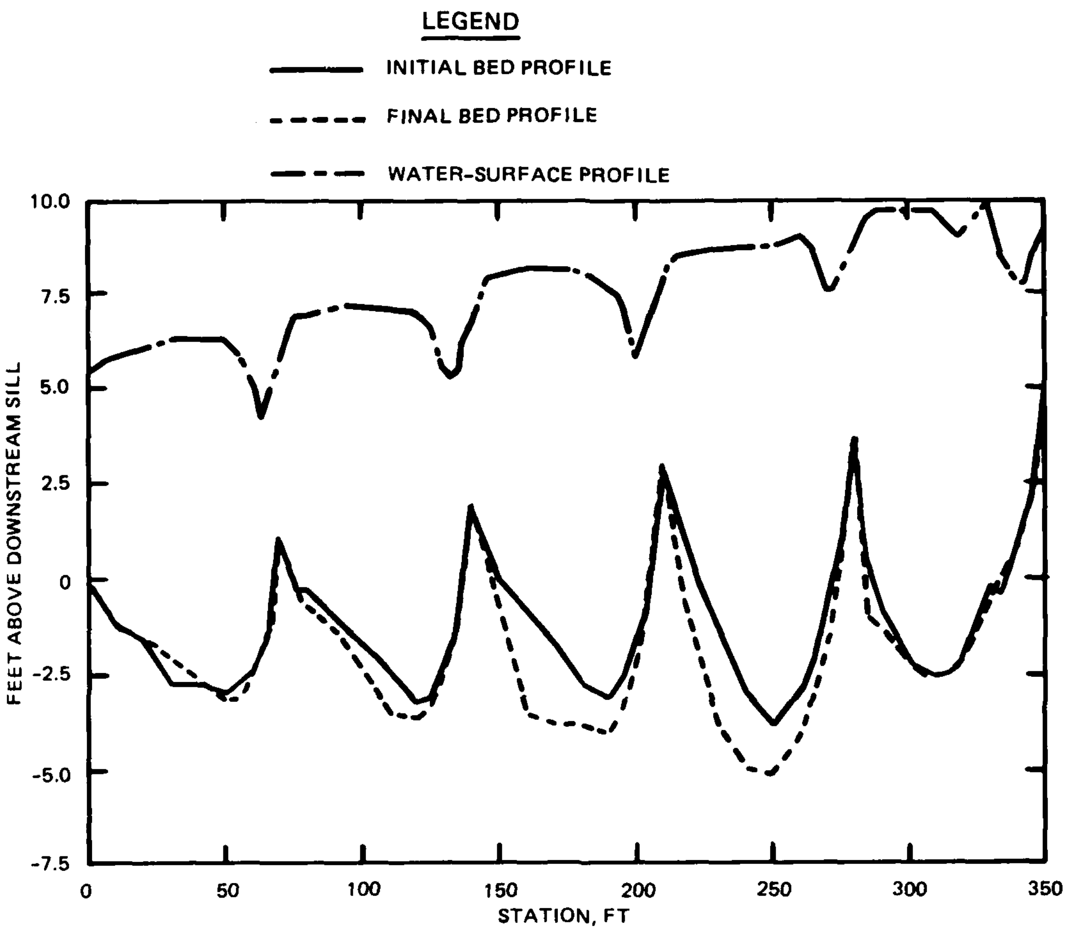
BED AND WATER-SURFACE PROFILES

UNPROTECTED CHANNEL

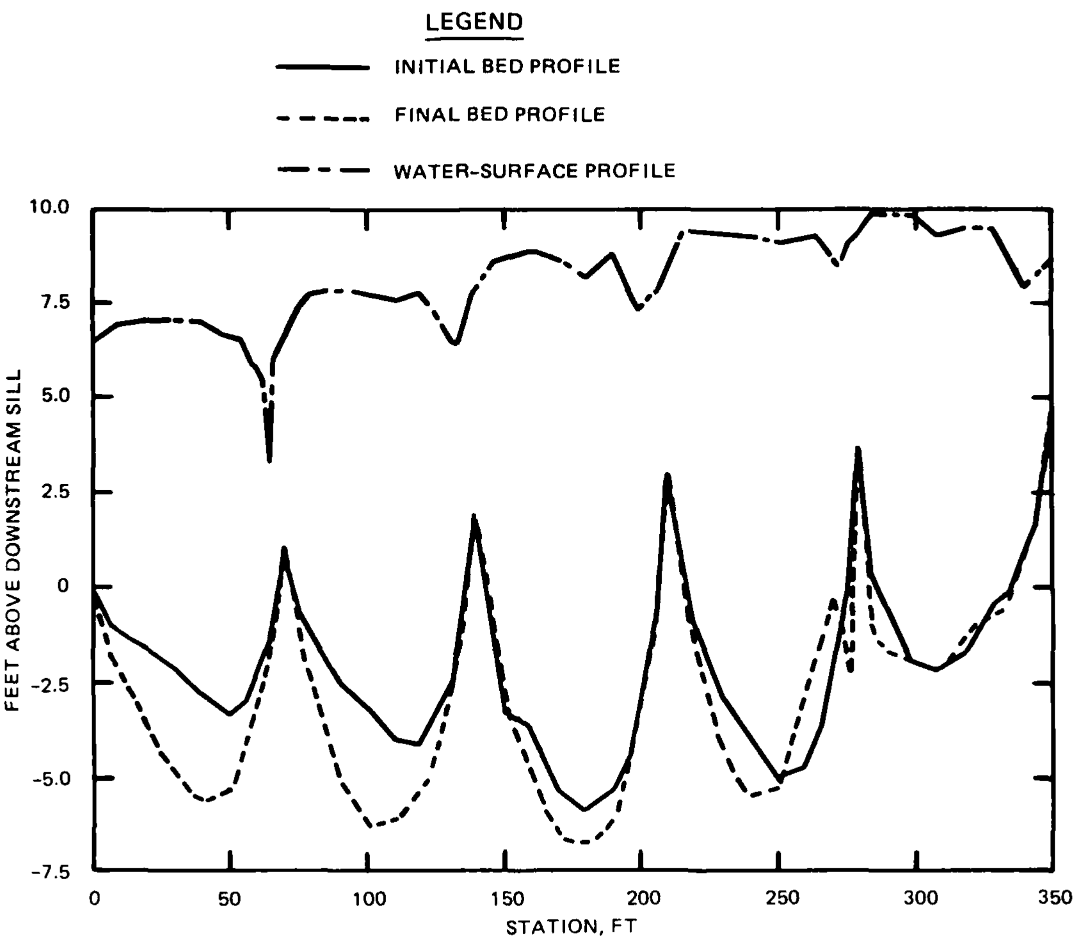
DISCHARGE 3,500 CFS



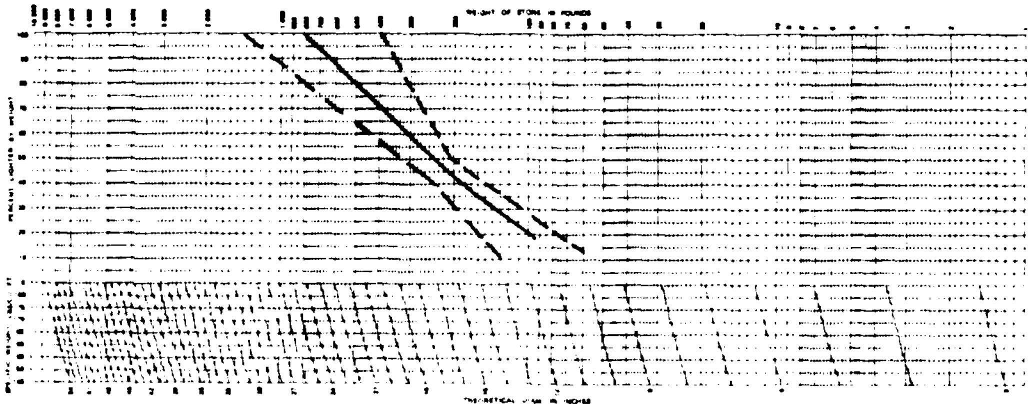
BED AND WATER-SURFACE PROFILES
UNPROTECTED CHANNEL
DISCHARGE 4,000 CFS



BED AND WATER-SURFACE PROFILES
UNPROTECTED CHANNEL
DISCHARGE 4,500 CFS

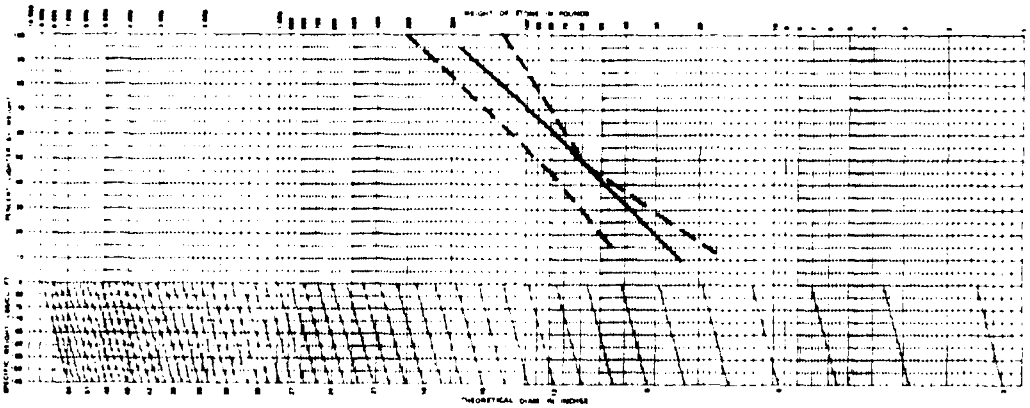


BED AND WATER-SURFACE PROFILES
UNPROTECTED CHANNEL
DISCHARGE 5,000 CFS



CLASS A RIPRAP

W_{50} = 200 LB



CLASS B RIPRAP

W_{50} = 60 LB

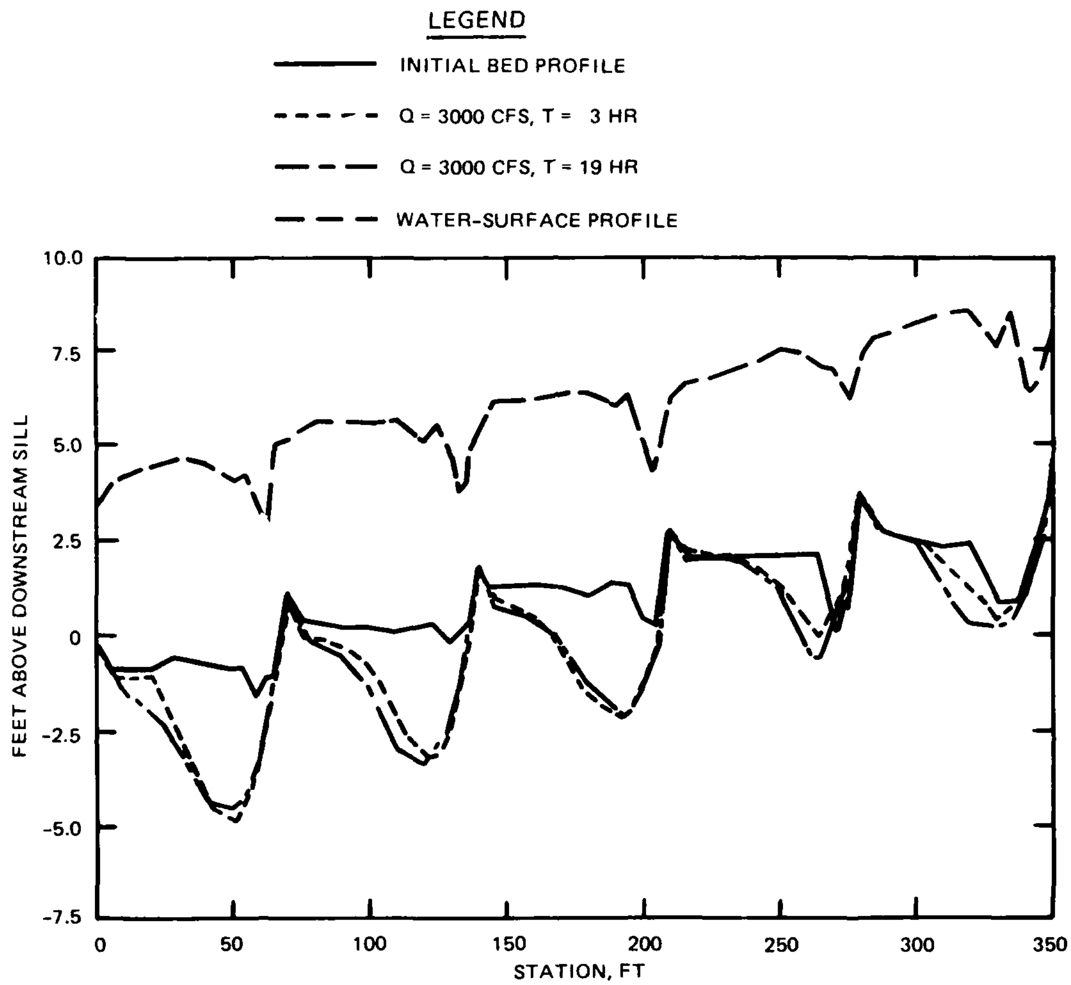
LEGEND

– – – – GRADATION LIMITS

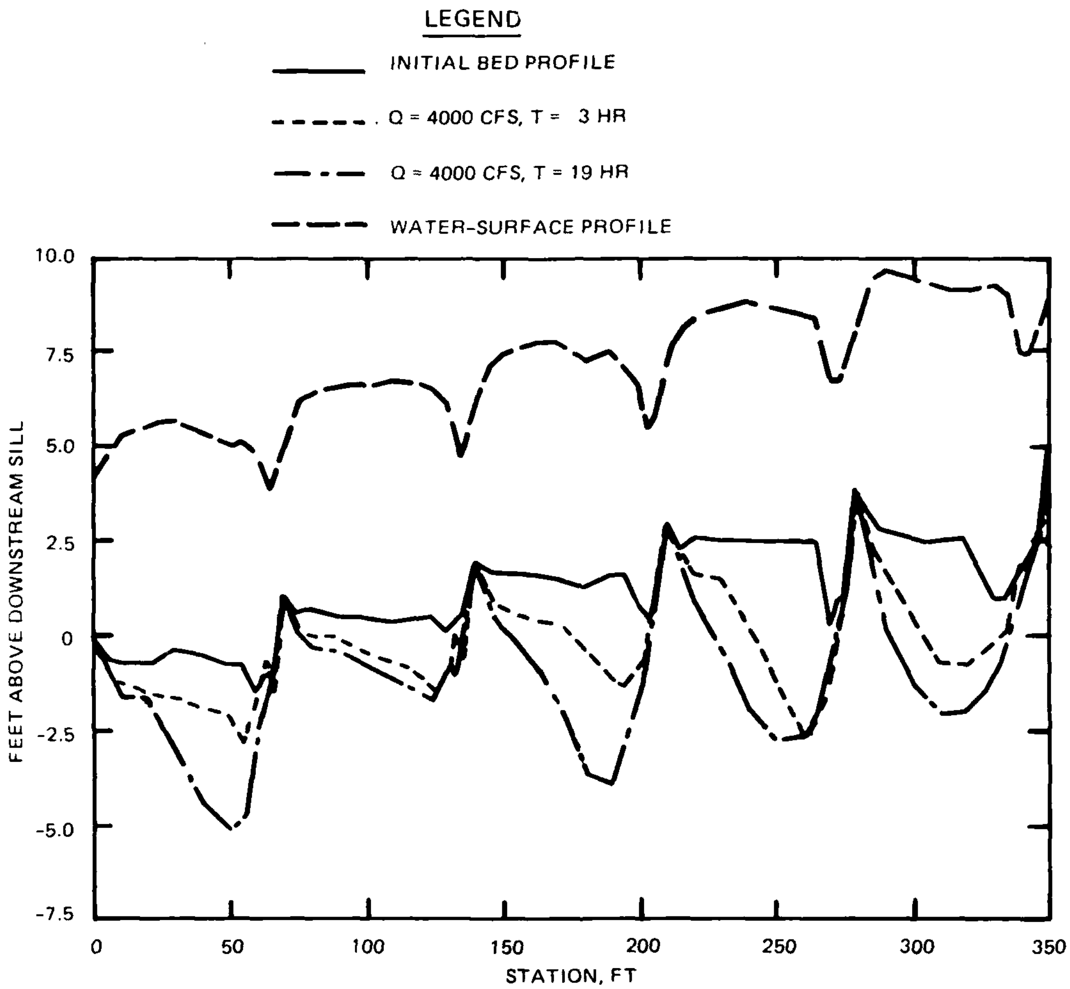
——— SIMULATED IN MODEL

RIPRAP GRADATION

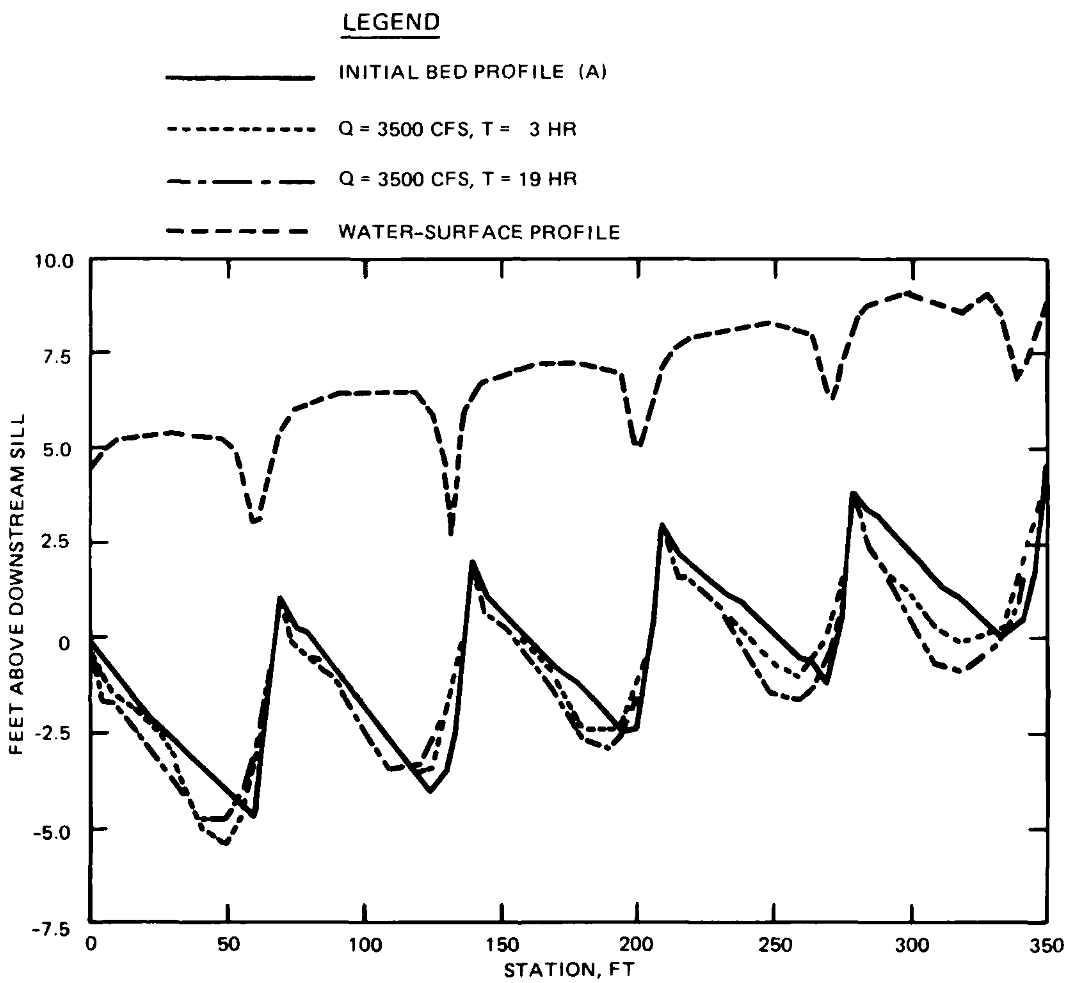
BY WEIGHT



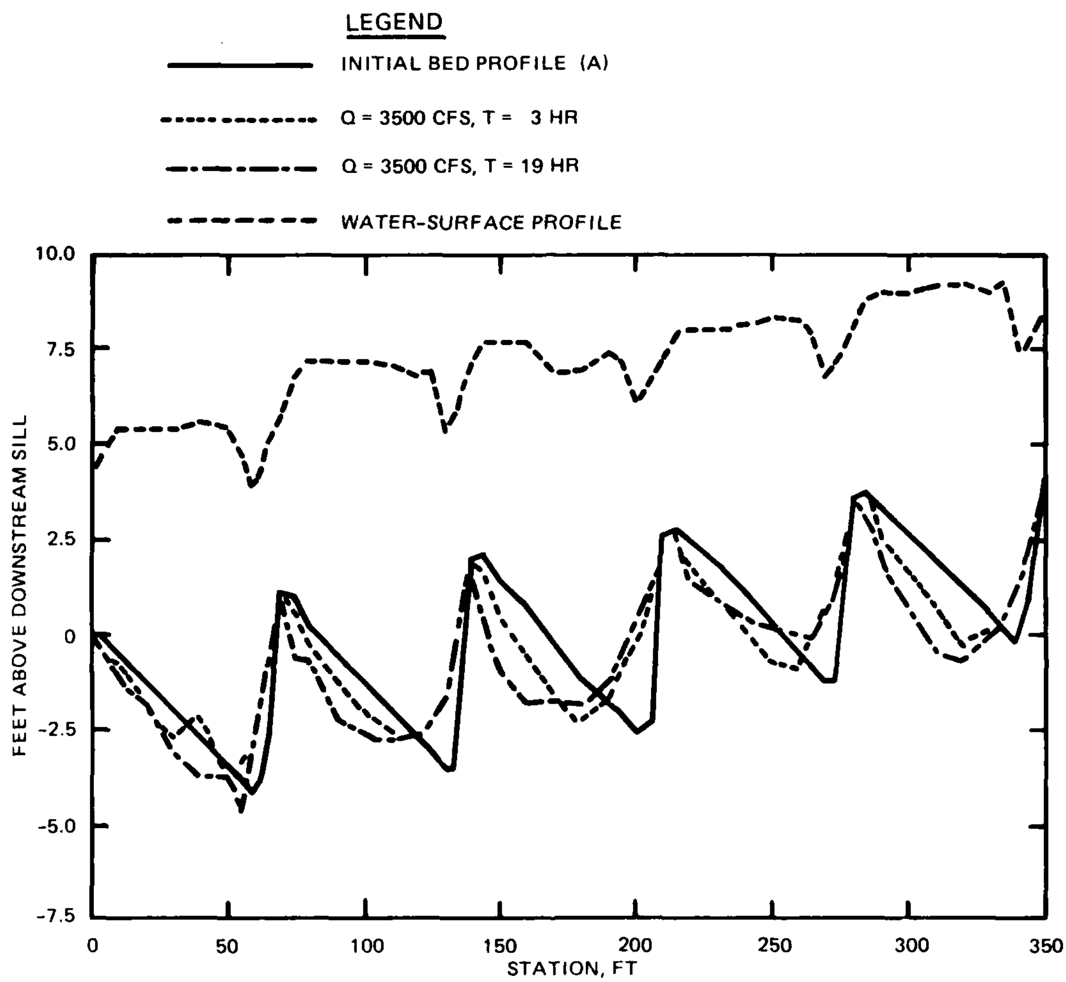
BED AND WATER-SURFACE PROFILES
TYPE 1 (ORIGINAL) DESIGN
STABILIZER PROTECTION
70-FT-WIDE CHANNEL
DISCHARGE 3.000 CFS



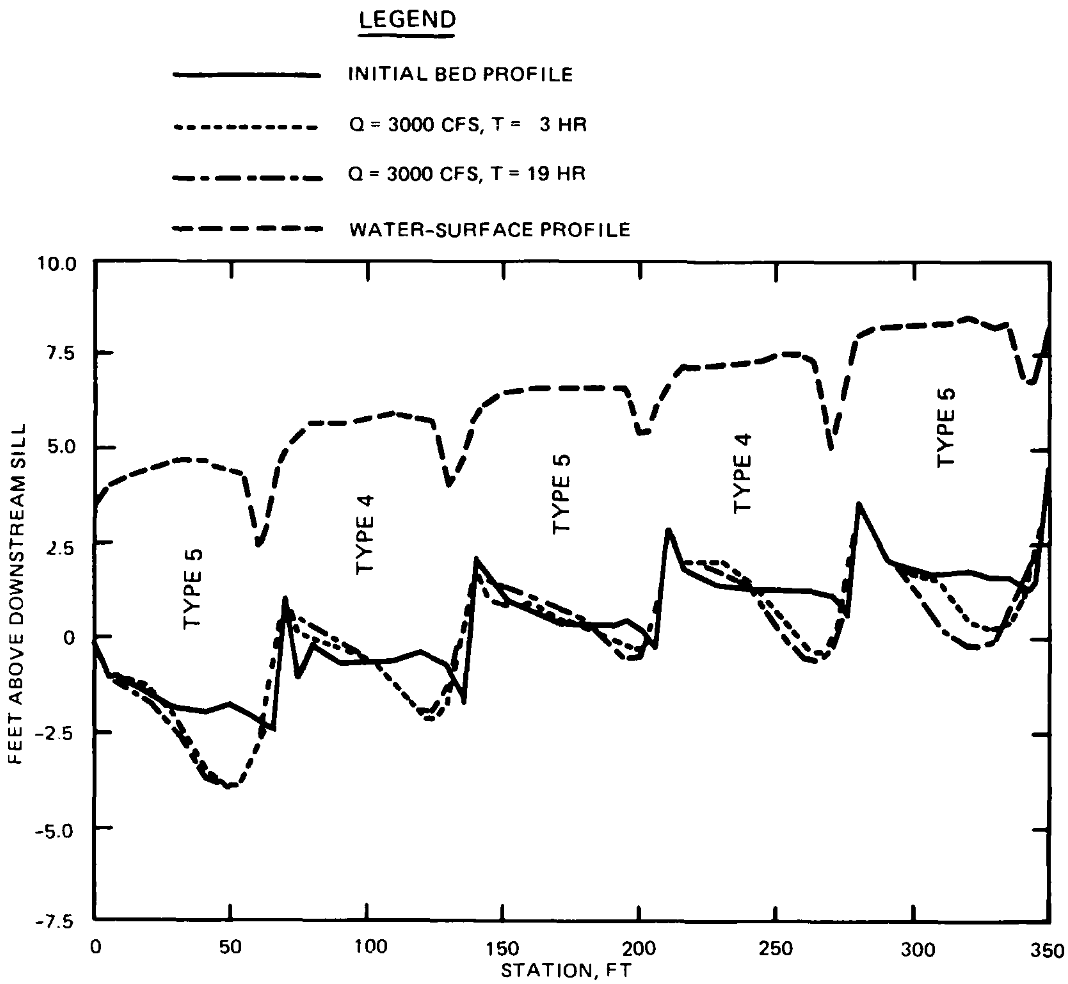
BED AND WATER-SURFACE PROFILES
TYPE 1 (ORIGINAL) DESIGN
STABILIZER PROTECTION
70-FT-WIDE CHANNEL
DISCHARGE 4,000 CFS



BED AND WATER-SURFACE PROFILES
TYPE 2 DESIGN
STABILIZER PROTECTION
70-FT-WIDE CHANNEL
DISCHARGE 3,500 CFS

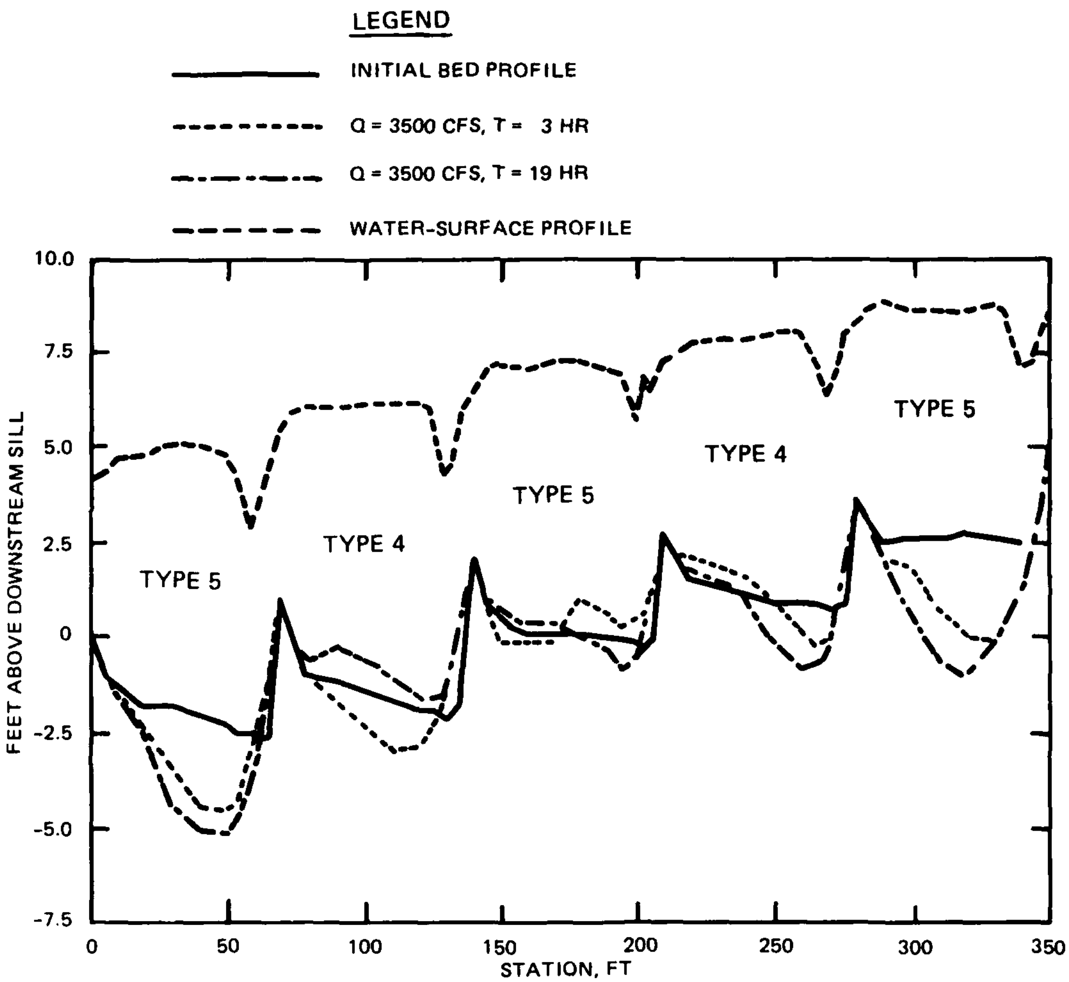


BED AND WATER-
SURFACE PROFILES
TYPE 3 DESIGN
STABILIZER PROTECTION
70-FT-WIDE CHANNEL
DISCHARGE 3,500 CFS

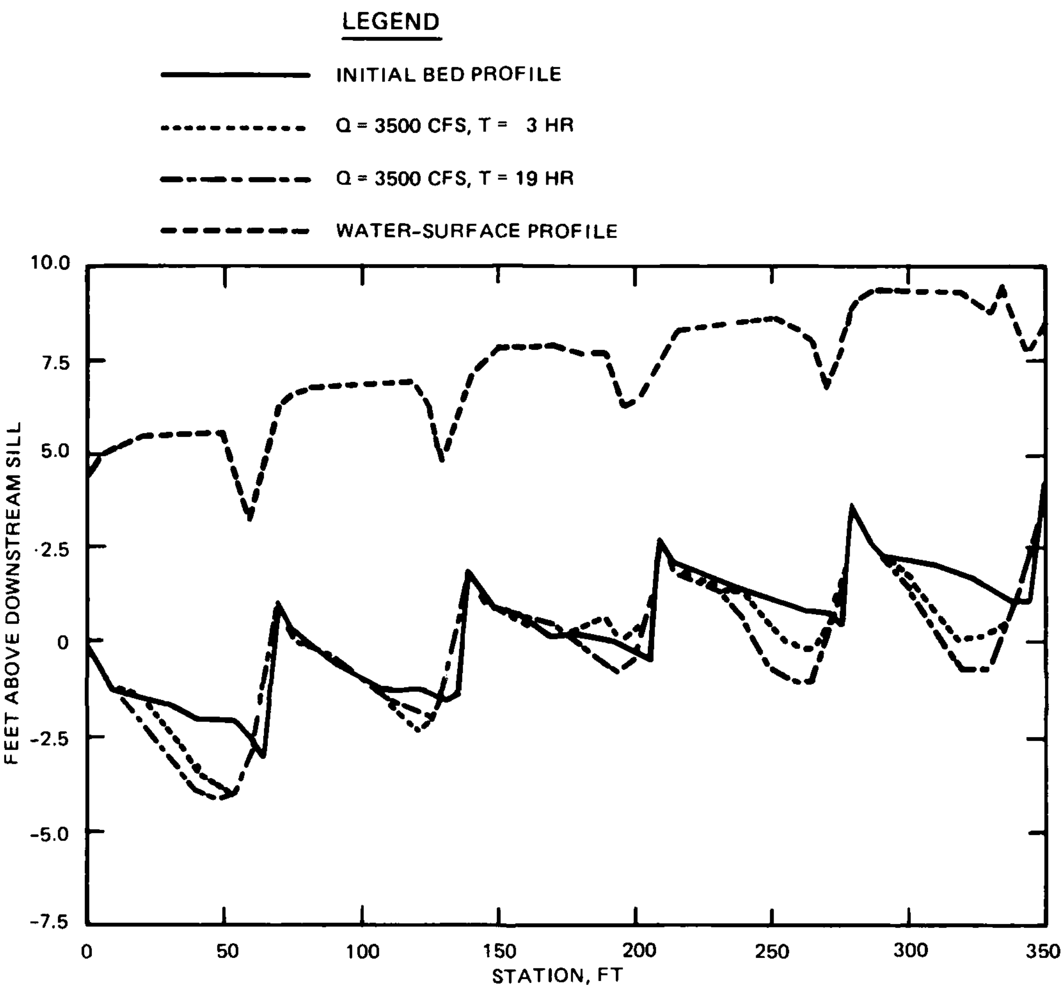


BED AND WATER-SURFACE PROFILES
TYPES 4 AND 5 DESIGNS
STABILIZER PROTECTION
70-FT-WIDE CHANNEL
DISCHARGE 3,000 CFS

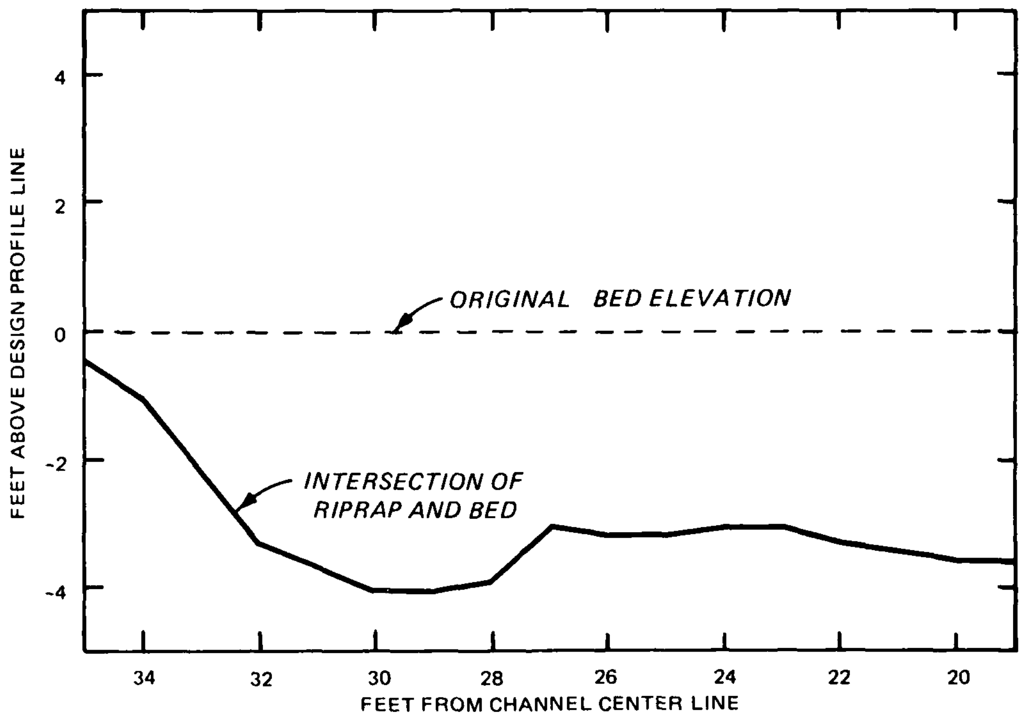
PLATE 16



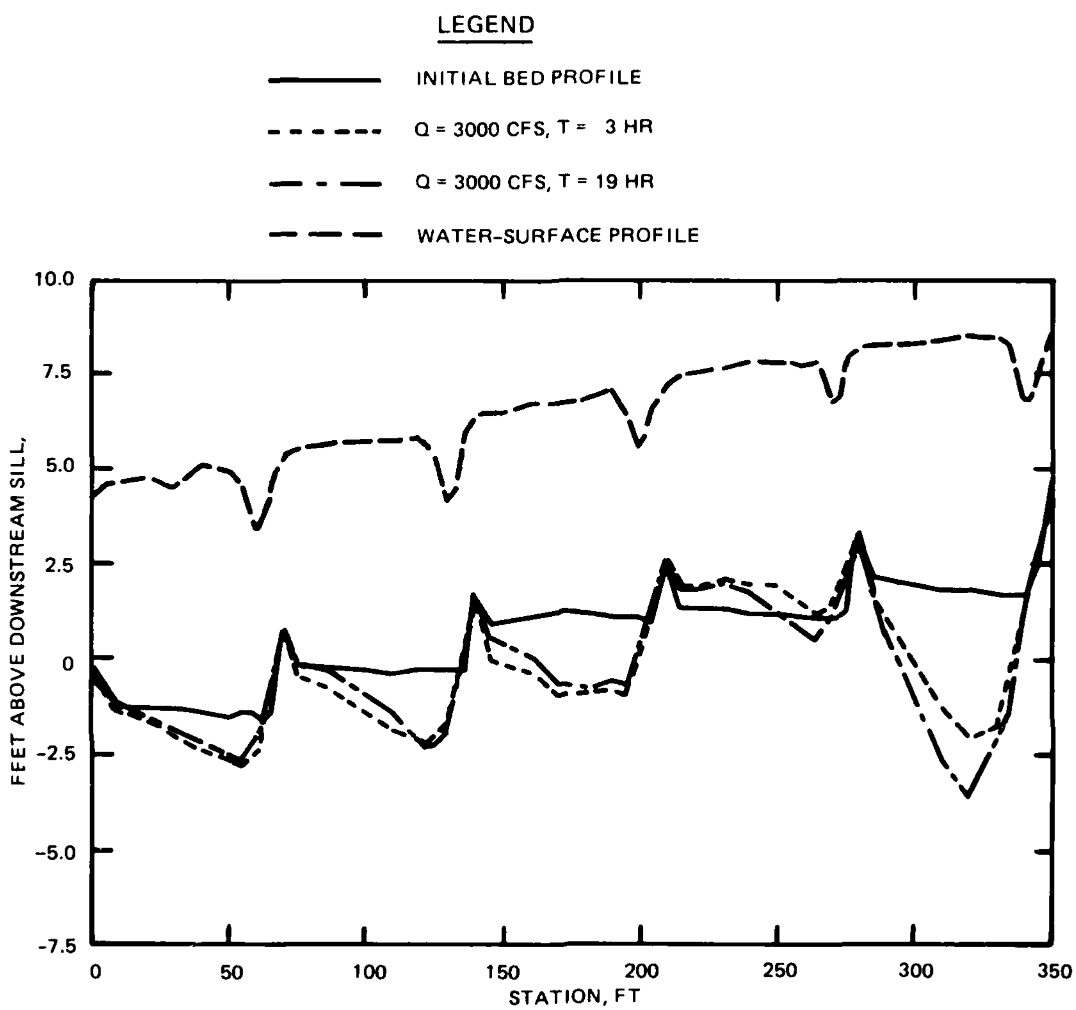
BED AND WATER-SURFACE PROFILES
TYPES 4 AND 5 DESIGNS
STABILIZER PROTECTION
70-FT-WIDE CHANNEL
DISCHARGE 3,500 CFS



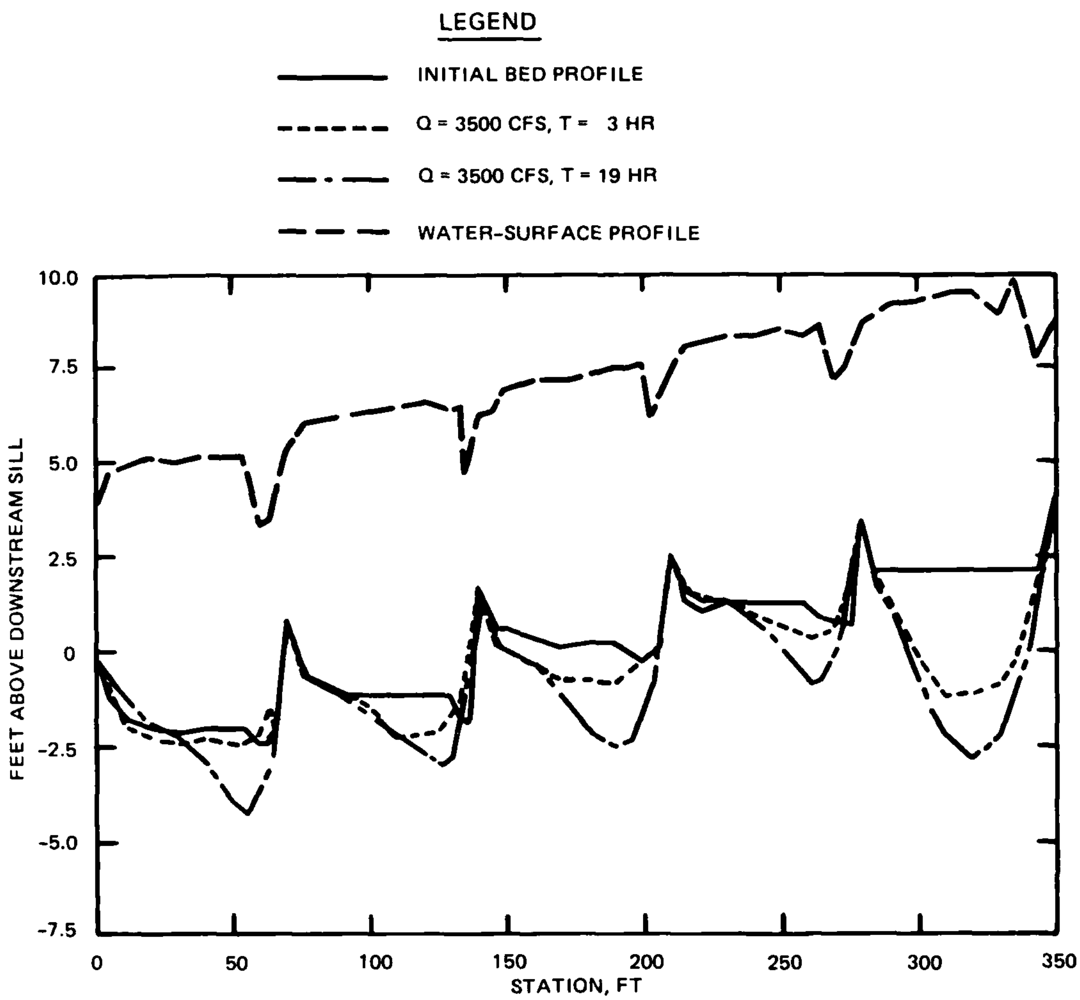
BED AND WATER-SURFACE PROFILES
TYPE 6 DESIGN
STABILIZER PROTECTION
70-FT-WIDE CHANNEL
DISCHARGE 3,500 CFS



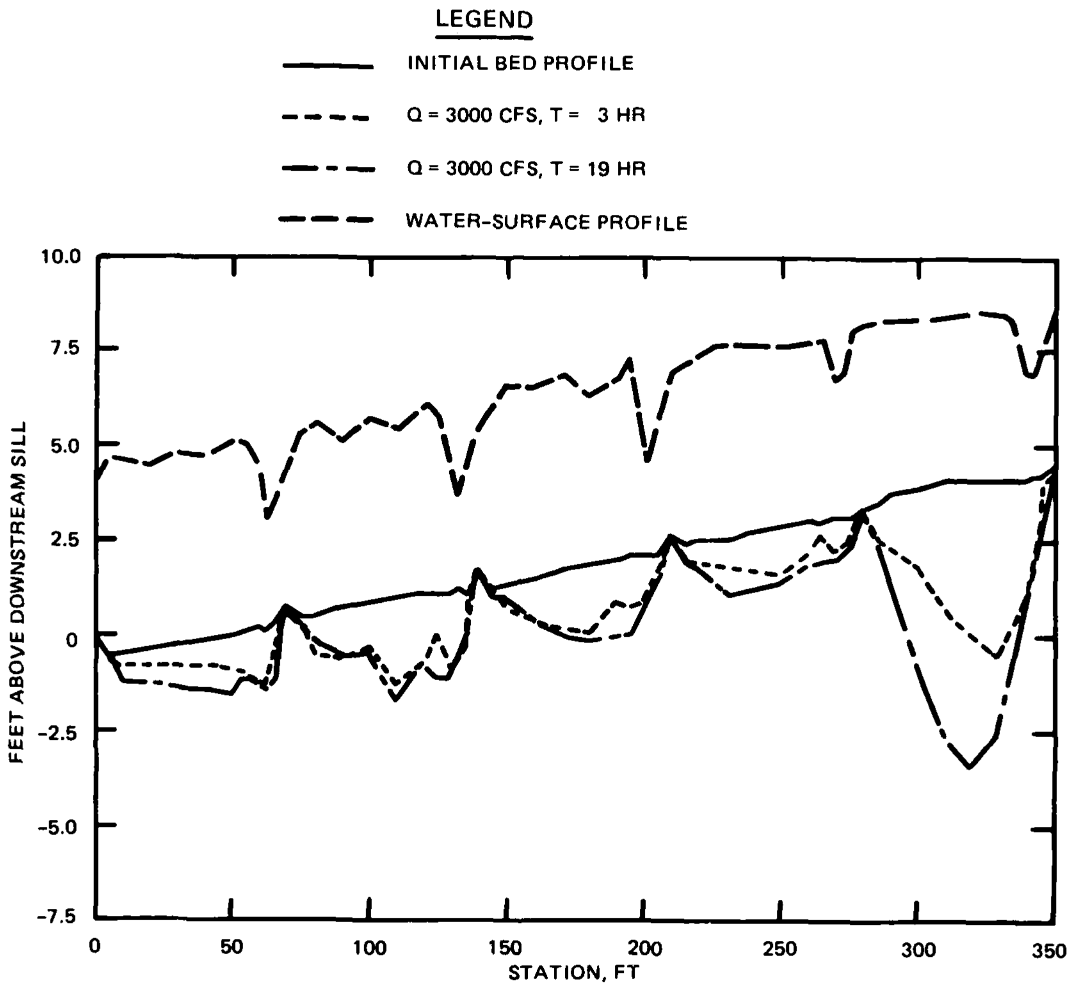
CROSS SECTION
OF CHANNEL SCOUR
TYPE 4 DESIGN
STABILIZER PROTECTION AND
TYPE 1 (ORIGINAL) DESIGN
BANK PROTECTION 25 FT
DOWNSTREAM OF STABILIZER
DISCHARGE 3,500 CFS



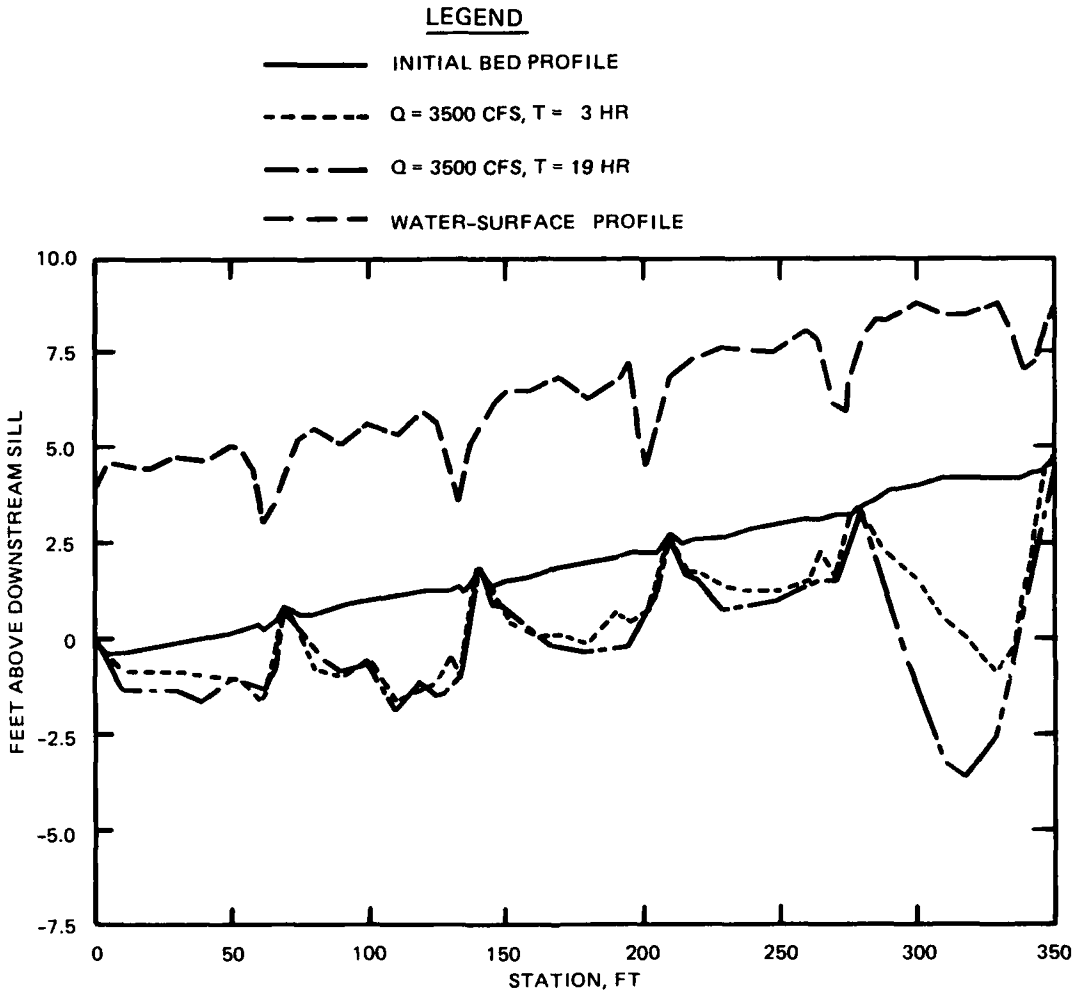
BED AND WATER-SURFACE PROFILES
TYPE 4 DESIGN
STABILIZER PROTECTION
AND TYPE 1 (ORIGINAL)
DESIGN BANK PROTECTION
70-FT-WIDE CHANNEL
DISCHARGE 3,000 CFS



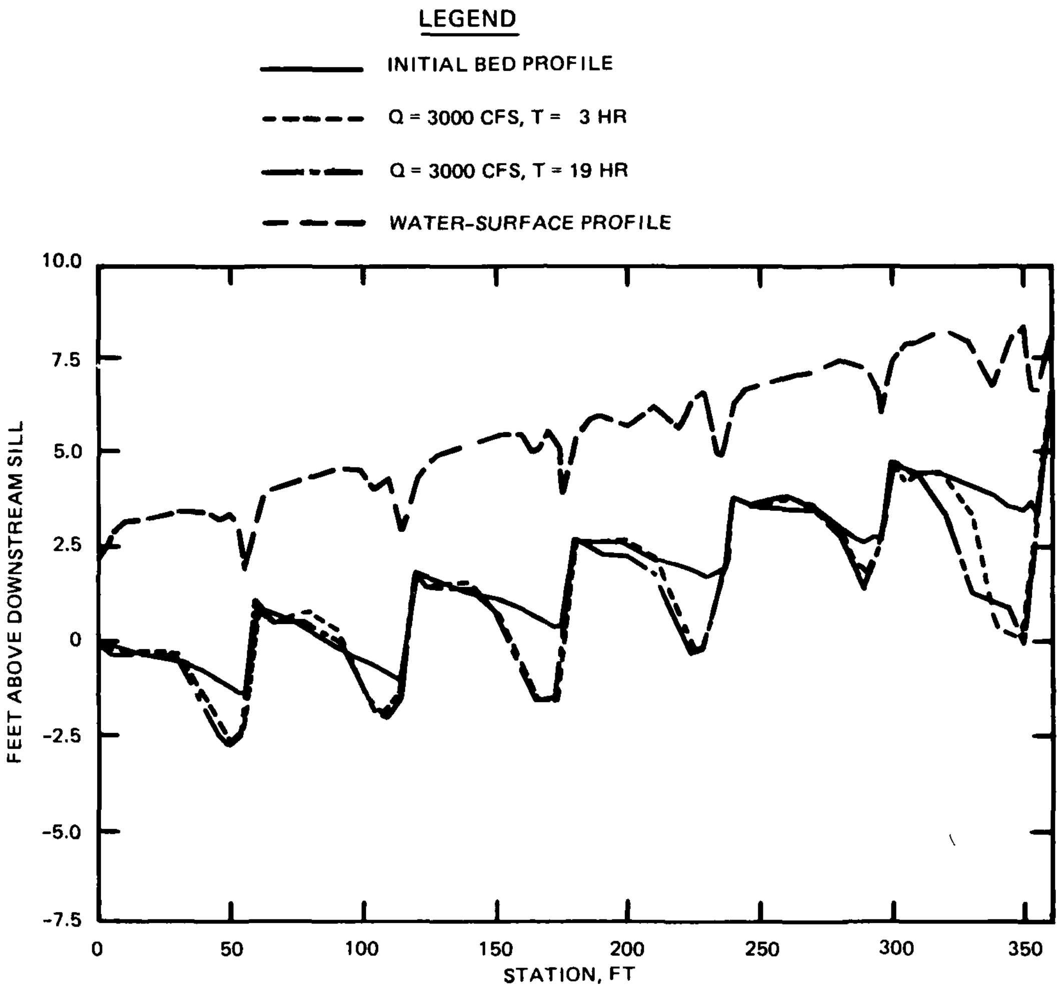
BED AND WATER-SURFACE PROFILES
TYPE 4 DESIGN
STABILIZER PROTECTION
AND TYPE 1 (ORIGINAL)
DESIGN BANK PROTECTION
70-FT-WIDE CHANNEL
DISCHARGE 3,500 CFS



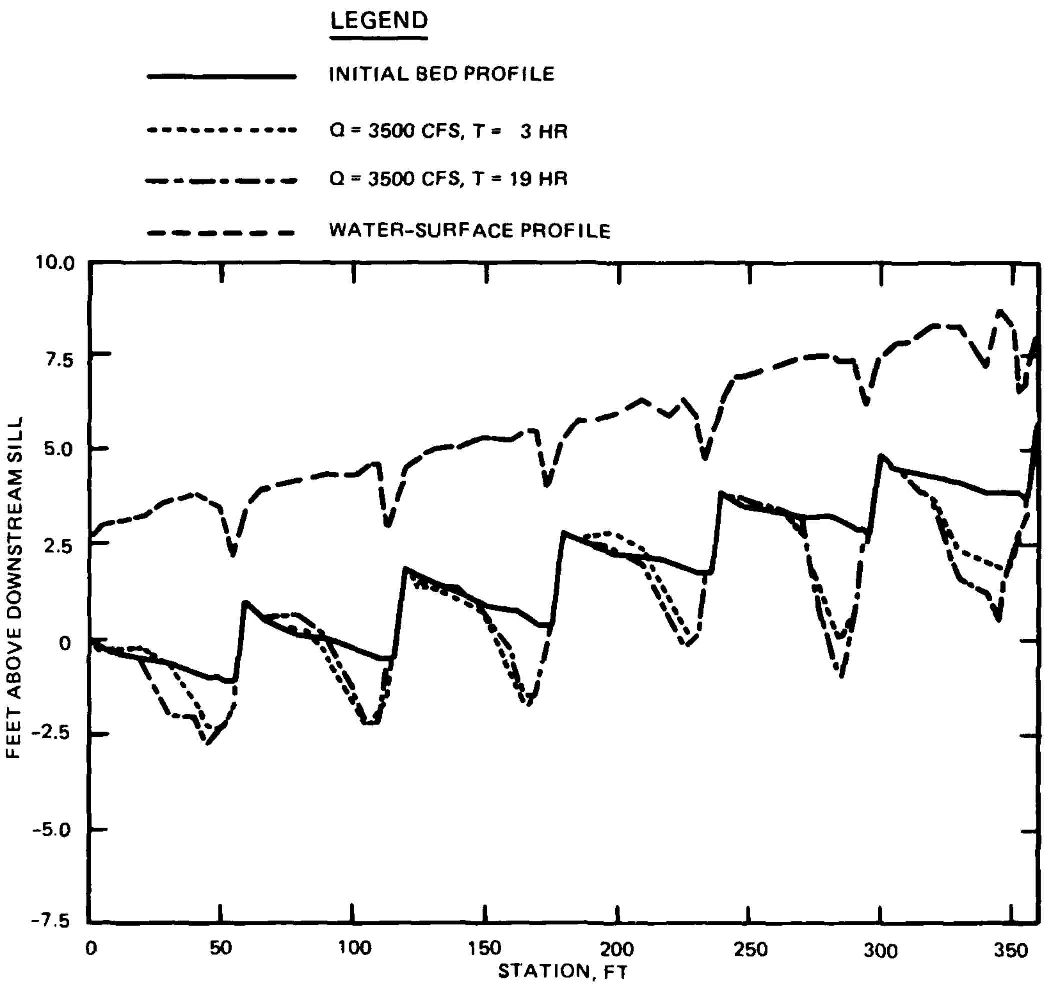
BED AND WATER-SURFACE PROFILES
TYPE 2 DESIGN
STABILIZER PROTECTION
AND TYPE 1 (ORIGINAL)
DESIGN BANK PROTECTION
70-FT-WIDE CHANNEL
DISCHARGE 3,000 CFS



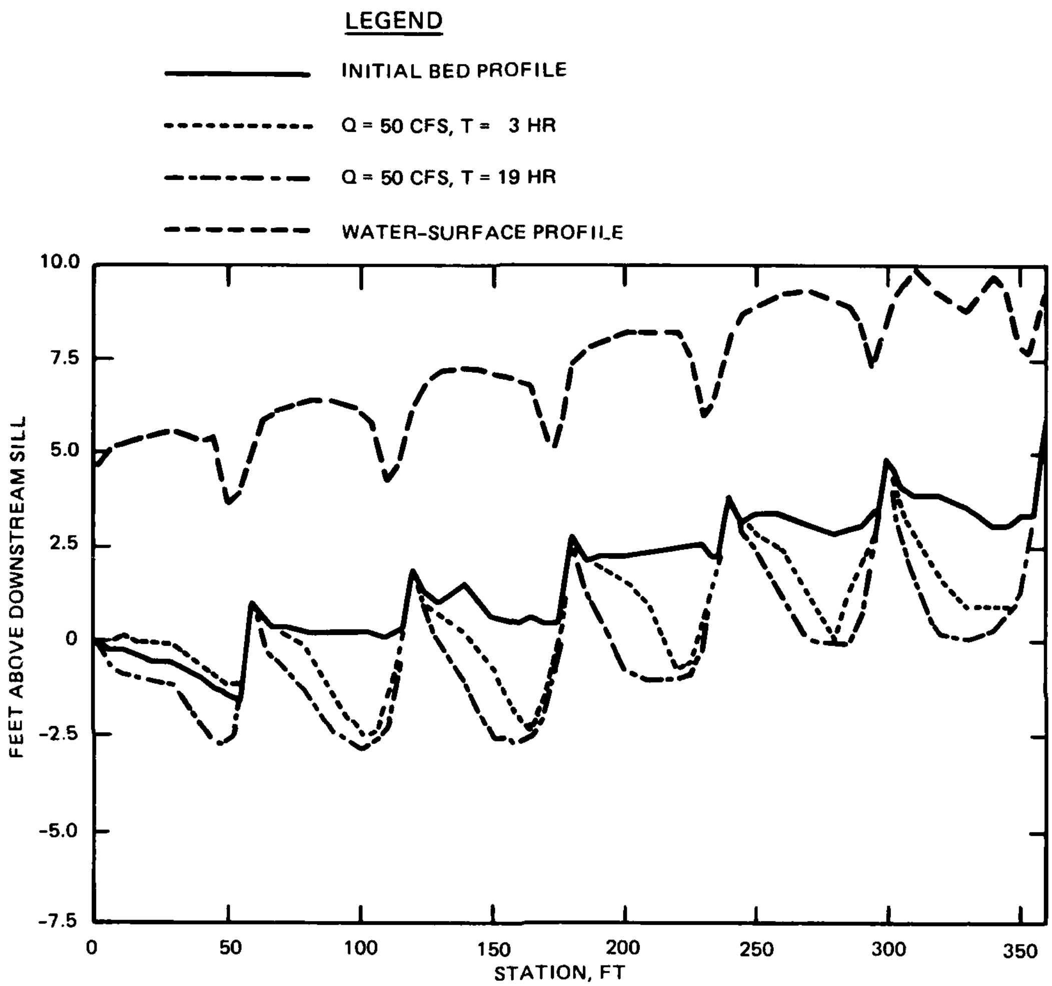
BED AND WATER-SURFACE PROFILES
TYPE 2 DESIGN
STABILIZER PROTECTION
AND TYPE 1 (ORIGINAL)
DESIGN BANK PROTECTION
70-FT-WIDE CHANNEL
DISCHARGE 3,500 CFS



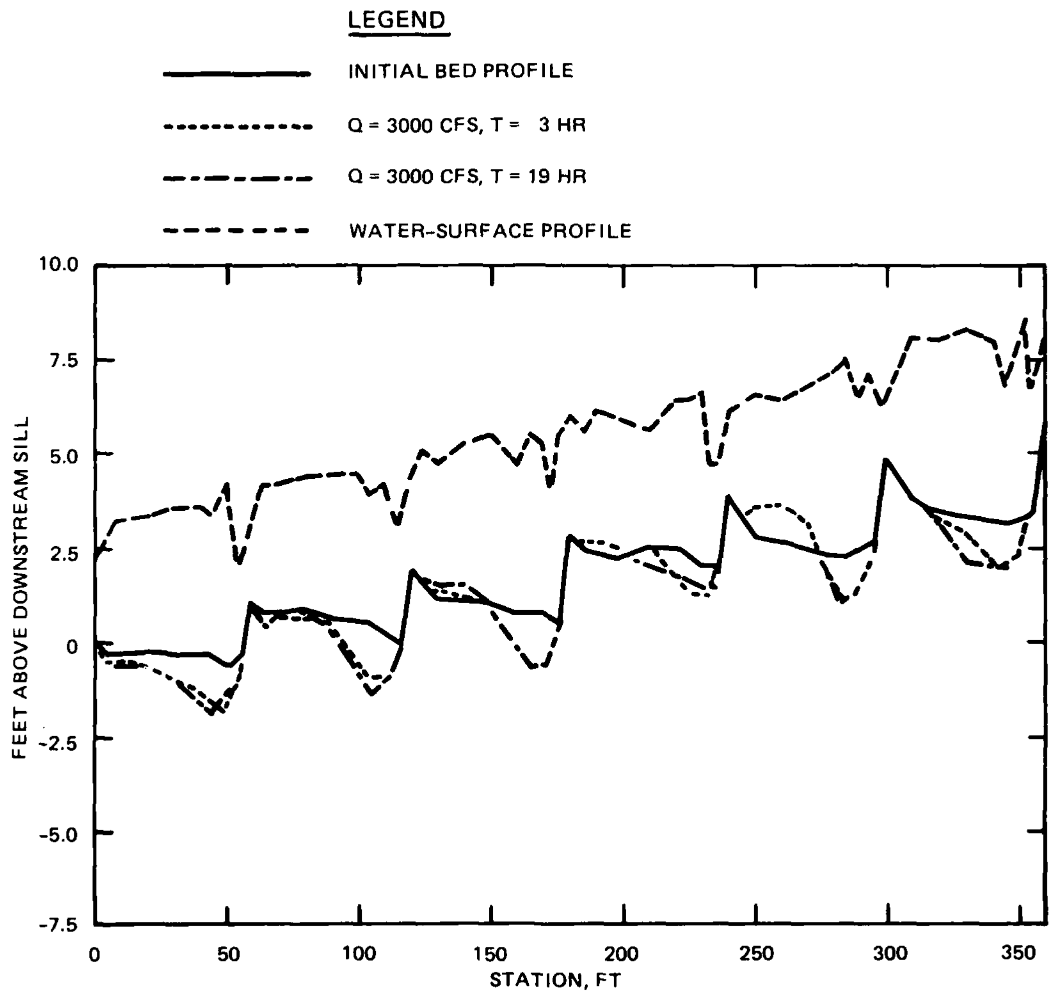
BED AND WATER-SURFACE PROFILES
TYPE 2 DESIGN
STABILIZER PROTECTION
120-FT-WIDE CHANNEL
DISCHARGE 3,000 CFS



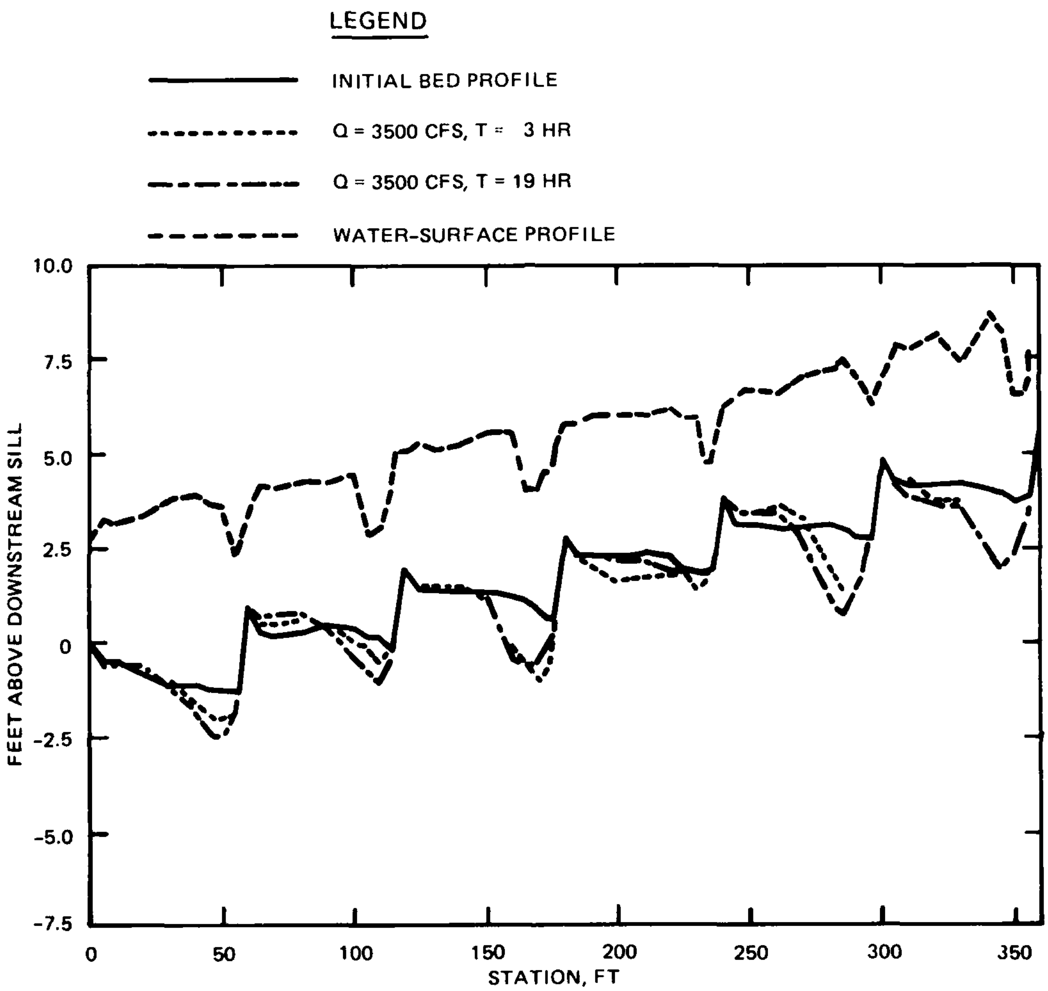
BED AND WATER-
SURFACE PROFILES
TYPE 2 DESIGN
STABILIZER PROTECTION
120-FT-WIDE CHANNEL
DISCHARGE 3,500 CFS



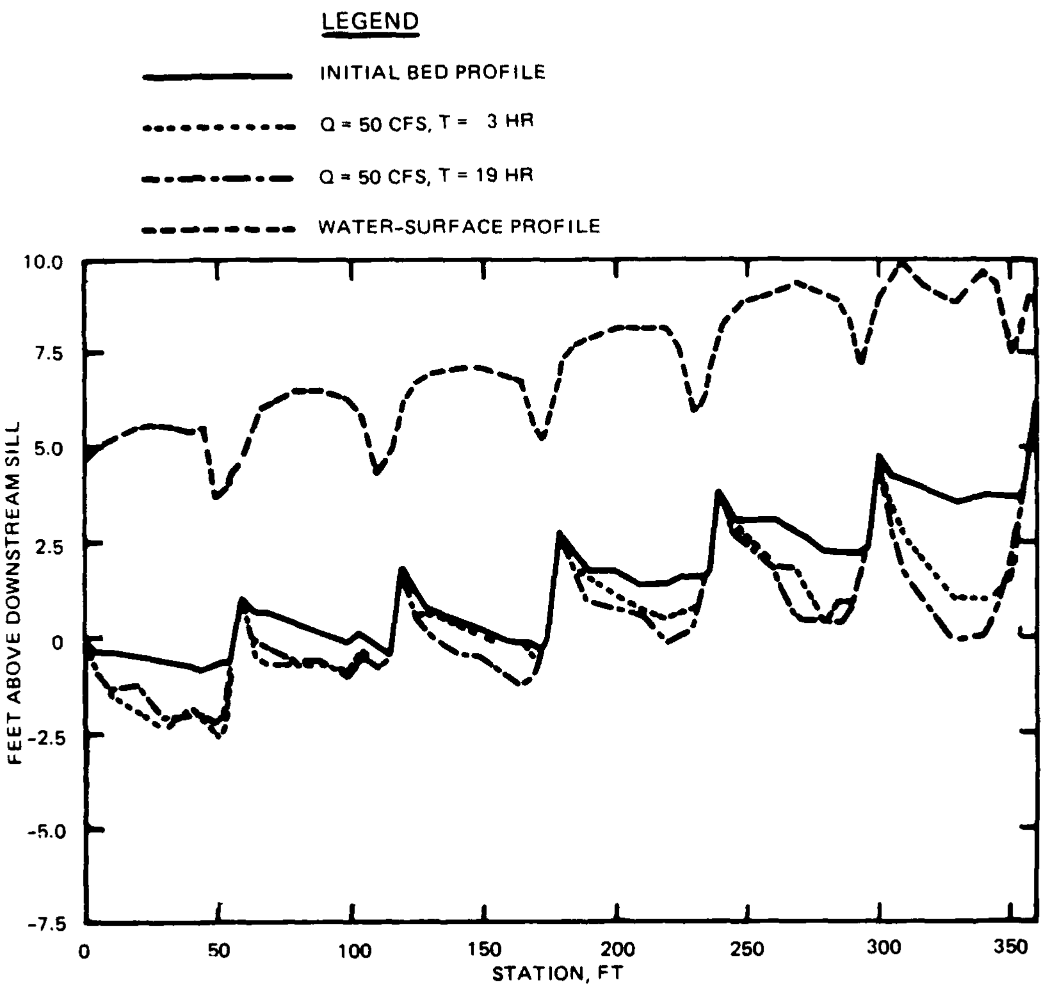
BED AND WATER-SURFACE PROFILES
TYPE 2 DESIGN
STABILIZER PROTECTION
120-FT-WIDE CHANNEL
UNIT DISCHARGE 50 CFS/FT



BED AND WATER-SURFACE PROFILES
TYPE 2 DESIGN STABILIZER PROTECTION AND TYPE 2 DESIGN BANK PROTECTION
120-FT-WIDE CHANNEL
DISCHARGE 3,000 CFS



BED AND WATER-SURFACE PROFILES
TYPE 2 DESIGN STABILIZER PROTECTION AND TYPE 2 DESIGN BANK PROTECTION
120-FT-WIDE CHANNEL
DISCHARGE 3,500 CFS



BED AND WATER-SURFACE PROFILES
TYPE 2 DESIGN AND STABILIZER PROTECTION AND TYPE 2 DESIGN BANK PROTECTION
120-FT-WIDE CHANNEL
UNIT DISCHARGE 50 CFS/FT

END

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